

U.S. DEPARTMENT OF COMMERCE
National Technical Information Service

PB-253 410

Urban Stormwater Runoff: Determination of Volumes and Flowrates

Illinois Univ. at Urbana-Champaign

Prepared For
Municipal Environmental Research Lab.

May 1976

KEEP UP TO DATE

Between the time you ordered this report—which is only one of the hundreds of thousands in the NTIS information collection available to you—and the time you are reading this message, several *new* reports relevant to your interests probably have entered the collection.

Subscribe to the **Weekly Government Abstracts** series that will bring you summaries of new reports as soon as they are received by NTIS from the originators of the research. The WGA's are an NTIS weekly newsletter service covering the most recent research findings in 25 areas of industrial, technological, and sociological interest—invaluable information for executives and professionals who must keep up to date.

The executive and professional information service provided by NTIS in the **Weekly Government Abstracts** newsletters will give you thorough and comprehensive coverage of government-conducted or sponsored re-

search activities. And you'll get this important information within two weeks of the time it's released by originating agencies.

WGA newsletters are computer produced and electronically photocomposed to slash the time gap between the release of a report and its availability. You can learn about technical innovations immediately—and use them in the most meaningful and productive ways possible for your organization. Please request NTIS-PR-205/PCW for more information.

The weekly newsletter series will keep you current. But *learn what you have missed in the past* by ordering a computer **NTISearch** of all the research reports in your area of interest, dating as far back as 1964, if you wish. Please request NTIS-PR-186/PCN for more information.

WRITE: Managing Editor
5285 Port Royal Road
Springfield, VA 22161

Keep Up To Date With SRIM

SRIM (Selected Research in Microfiche) provides you with regular, automatic distribution of the complete texts of NTIS research reports *only* in the subject areas you select. SRIM covers almost all Government research reports by subject area and/or the originating Federal or local government agency. You may subscribe by any category or subcategory of our WGA (**Weekly Government Abstracts**) or **Government Reports Announcements and Index** categories, or to the reports issued by a particular agency such as the Department of Defense, Federal Energy Administration, or Environmental Protection Agency. Other options that will give you greater selectivity are available on request.

The cost of SRIM service is only 45¢ domestic (60¢ foreign) for each complete

microfiched report. Your SRIM service begins as soon as your order is received and processed and you will receive biweekly shipments thereafter. If you wish, your service will be backdated to furnish you microfiche of reports issued earlier.

Because of contractual arrangements with several Special Technology Groups, not all NTIS reports are distributed in the SRIM program. You will receive a notice in your microfiche shipments identifying the exceptionally priced reports not available through SRIM.

A deposit account with NTIS is required before this service can be initiated. If you have specific questions concerning this service, please call (703) 451-1558, or write NTIS, attention SRIM Product Manager.

This information product distributed by

NTIS

U.S. DEPARTMENT OF COMMERCE
National Technical Information Service
5285 Port Royal Road
Springfield, Virginia 22161

162125

EPA-600/2-76-116
May 1976

Environmental Protection Technology Series

PB 253 410

URBAN STORMWATER RUNOFF: DETERMINATION OF VOLUMES AND FLOWRATES



REPRODUCED BY
NATIONAL TECHNICAL
INFORMATION SERVICE
U. S. DEPARTMENT OF COMMERCE
SPRINGFIELD, VA. 22161

Municipal Environmental Research Laboratory
Office of Research and Development
U.S. Environmental Protection Agency
Cincinnati, Ohio 45268

TECHNICAL REPORT DATA (Please read instructions on the reverse before completing)		
1. REPORT NO. EPA-600/2-76-116	2.	3. RECIPIENT'S ACCESSION NO. PB253410
4. TITLE AND SUBTITLE URBAN STORMWATER RUNOFF: DETERMINATION OF VOLUMES AND FLOWRATES	5. REPORT DATE May 1976 (Issuing Date)	6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Ven Te Chow and Ben Chie Yen	8. PERFORMING ORGANIZATION REPORT NO.	
9. PERFORMING ORGANIZATION NAME AND ADDRESS Department of Civil Engineering University of Illinois at Urbana-Champaign Urbana, Illinois 61801	10. PROGRAM ELEMENT NO. PE.1RC611 (SOS#2, ID Code: 03)	11. CONTRACT/ ~~NUMBER~~ 68-03-0302
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268	13. TYPE OF REPORT AND PERIOD COVERED 6/73 - 9/75	14. SPONSORING AGENCY CODE EPA-ORD
15. SUPPLEMENTARY NOTES Project Officer: Hugh E. Masters, 342-7541		
16. ABSTRACT An investigation is made to (a) develop a method of depth-duration-frequency analysis for precipitation events having short return period (high frequency) for urban storm water runoff management and control purposes; (b) develop a new high accuracy urban stormwater runoff determination method which when verified, can be used for projects requiring high accuracy detailed runoff results and can also be used as the calibration scale for the less accurate urban runoff prediction methods; and (c) compare and evaluate selected urban stormwater runoff prediction methods. The eight methods evaluated are the rational method, unit hydrograph method, Chicago hydrograph method, British Transport and Road Research Laboratory method, University of Cincinnati Urban Runoff method, Dorsch Hydrograph Volume method, EPA Storm Water Management Model, and Illinois Urban Storm Runoff method. The comparison and evaluation is done by using four recorded hyetographs of the Oakdale Avenue Drainage Basin in Chicago to produce the predicted hydrographs by the methods and the results are compared with recorded hydrographs. The relative merits of the methods are discussed and recommendations are made. This report was submitted in fulfillment of Contract No. 68-03-0302 by the Department of Civil Engineering, University of Illinois at Urbana-Champaign, under sponsorship of the Office of Research and Development, U.S. Environmental Protection Agency. Work was completed as of September 25, 1975.		
17. KEY WORDS AND DOCUMENT ANALYSIS		
a. DESCRIPTORS *Drainage, *Rainfall, *Runoff, *Storm sewers, *Surface drainage, Computer programs, Environmental engineering, Hydraulics, Hydrology, Mathematical models, Probability theory, Sewers, Surface water runoff, Urbanization, Water pollution, Water quality, Water storage	b. IDENTIFIERS/OPEN ENDED TERMS Frequency analysis, Hydrographs, Oakdale Avenue (Chicago), Rainfall-runoff, Urban hydrology, Urban runoff modeling	c. COSATI Field/Group 13B
PRICES SUBJECT TO CHANGE		
18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS (This Report) Unclassified	21. NO. OF PAGES 253
	20. SECURITY CLASS (This page) Unclassified	22. PRICE 9.00 - 2.25

EPA-600/2-76-116

May 1976

URBAN STORMWATER RUNOFF:
DETERMINATION OF VOLUMES AND FLOWRATES

by

Ven Te Chow and Ben Chie Yen
Department of Civil Engineering
University of Illinois at Urbana-Champaign
Urbana, Illinois 61801

Contract No. 68-03-0302

Project Officer

Chi Yuan Fan
Storm and Combined Sewer Section
Municipal Environmental Research Laboratory (Cincinnati)
Edison, New Jersey 08817

MUNICIPAL ENVIRONMENTAL RESEARCH LABORATORY
OFFICE OF RESEARCH AND DEVELOPMENT
U.S. ENVIRONMENTAL PROTECTION AGENCY
CINCINNATI, OHIO 45268

DISCLAIMER

The Municipal Environmental Research Laboratory has reviewed this report and approved its publication. Approval does not signify that the contents necessarily reflect the views and policies of the U.S. Environmental Protection Agency, nor does mention of trade names or commercial products constitute endorsement or recommendation for use.

FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollution discharges from municipal and community sources, for the preservation and treatment of public drinking water supplies, and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research; a most vital communications link between the researcher and the user community.

A prerequisite for effective control of storm runoff pollution is a reliable method to predict the quantity of the storm runoff. The time distribution of storm runoff from an urban drainage system depends on the areal and temporal distributions of the intensity of the rainfall and the physical characteristics of the drainage system. Numerous methods have been proposed to evaluate urban runoff from rainfall. Many have been accepted for engineering applications while others need yet to be tested and verified. Therefore an investigation to evaluate the methods on a common basis would be a significant contribution to the recent efforts on pollution control.

Francis T. Mayo
Director
Municipal Environmental
Research Laboratory

ABSTRACT

An investigation is made to (a) develop a method of depth-duration-frequency analysis for precipitation events having short return period (high frequency) for urban storm water runoff management and control purposes and (b) compare and evaluate selected urban storm water runoff prediction methods. The eight methods evaluated are the rational method, unit hydrograph method, Chicago hydrograph method, British Transport and Road Research Laboratory method, University of Cincinnati Urban Runoff method, Dorsch Hydrograph Volume method, EPA Storm Water Management Model, and Illinois Urban Storm Runoff method. The comparison and evaluation is done by using four recorded hyetographs of the Oakdale Avenue Drainage Basin in Chicago to produce the predicted hydrographs by the methods and the results are compared with recorded hydrographs. The relative merits of the methods are discussed and recommendations are made.

This report is submitted in fulfillment of Contract No. 68-03-0302 by the Department of Civil Engineering, University of Illinois at Urbana-Champaign, under sponsorship of the Office of Research and Development, U.S. Environmental Protection Agency. Work was completed as of September 25, 1975.

TABLE OF CONTENTS

	<u>Page</u>
Foreword	iii
Abstract	iv
List of Figures	vii
List of Tables	xi
Acknowledgments	xii
 <u>Sections</u>	
I. Summary and Conclusions	1
II. Recommendations	3
III. Introduction	6
1. Problems of urban storm water runoff	6
2. Objectives and scope of study	7
IV. Precipitation Analysis	9
1. Rainfall depth-duration-frequency analysis for urban runoff	9
2. Infiltration and other abstractions	39
3. Snow melt	41
V. Characteristics of Oakdale Avenue Basin	43
1. Surface drainage pattern	43
2. Sewer system	53
VI. Surface Runoff Model	55
1. Runoff in subcatchments	55
2. Gutter flow routing	63
3. Inlets	64
4. Computer program description	66
VII. Sewer System Routing Model	72
1. Sewer network representation	72
2. Method of solution	73
3. Computer program description	77

	<u>Page</u>
VIII. Water Quality Model	81
1. Water quality model formulation	81
2. Computer program description for surface runoff quality	83
3. Computer program description for Illinois Sewer System Water Quality Model	84
IX. General Description of Methods Evaluated	87
1. The rational method	87
2. Unit hydrograph method	90
3. Chicago hydrograph method	93
4. Transport and Road Research Laboratory Method	95
5. University of Cincinnati Urban Runoff Model	97
6. Dorsch Hydrograph-Volume-Method	100
7. EPA Storm Water Management Model	103
8. Illinois Urban Storm Runoff method	107
X. Evaluation of Methods	109
1. Hydrographs for the Eight Methods	109
2. Comparison of the methods	126
XI. References	130
XII. Notation and Abbreviations	136
Appendices	
A. Description and Listing of Computer Program for Frequency Analysis of Hourly Precipitation Data	142
B. Data Preparation and Listing of Computer Program for Illinois Surface Runoff Model	187
C. Data Preparation and Listing of Computer Program for Illinois Sewer System Water Quality Model	217
D. Application of Illinois Urban Storm Runoff Method to a Hypothetical Basin	229

FIGURES

<u>No.</u>		<u>Page</u>
1	Example Hyetograph	13
2	Trapezoidal Representation of Hyetograph	13
3	Probability Distribution of Rainstorm Depth	17
4	Probability Distribution of Rainstorm Duration	18
5	Probability Distribution of Average Rainstorm Intensity	19
6	Probability Distribution of Elapse Time Between Rainstorms	20
7	Conditional Distributions of Average Rainstorm Intensity	23-26
8	Gamma Density Function Parameters as Functions of Rainstorm Duration	32
9	Exponential Density Function Parameter as Function of Rainstorm Duration	33
10	Non-Exceedance Probability for Rainstorm Depth	35
11	Non-Exceedance Probability for Elapse Time Between Rainstorms	35
12	Conditional Distribution of Rainstorm Duration	37
13	Conditional Distribution of First Moment Arm of Hyetograph for Two-Hour Duration and for All Rainstorm Depth	38
14	Oakdale Basin Location Map	44
15	Typical Photographic View of Oakdale Basin	45
16	Drainage Pattern of Oakdale Basin	46
17	Schematic Drawing of Gutter Cross Section	48
18	Details of Grate Inlets	50-51
19	Circular Sewer Flow Cross Section	53
20	Block Diagram of Illinois Urban Storm Runoff Method	56
21	Definition Sketch of Open-Channel Flow	58
22	Evaluation of Weisbach Resistance Coefficient	60

<u>No.</u>		<u>Page</u>
23	Computational Grid for Semi-Implicit Four-Point Backward Difference Scheme	60
24	Identification of Types of Gutter and Inlet	68
25	Flow Chart for Illinois Surface Runoff Model Computer Program	69
26	Composition of Illinois Surface Runoff Model Computer Program	70
27	Solution by Method of Overlapping Y-Segments	76
28	Flow Chart for ISS Model Computer Program Flow Prediction Option	78
29	Flow Chart for Illinois Sewer System Water Quality Model Computer Program	85
30	Schematic Diagram of the Rational Method for Peak Discharge Prediction	91
31	Schematic Diagram of Unit Hydrograph Method	92
32	Schematic Diagram of Chicago Hydrograph Method	94
33	Schematic Diagram of TRRL Method	96
34	Schematic Diagram of Cincinnati Urban Runoff Model	99
35	Schematic Diagram of Structure of Dorsch Hydrograph-Volume-Method	102
36	Schematic Diagram of Water Quantity Routing of SWMM	106
37	Hyetograph and Hydrographs for May 19, 1959 Rainstorm	110
38	Hyetograph and Hydrographs for July 2, 1960 Rainstorm	111
39	Hyetograph and Hydrographs for April 29, 1963 Rainstorm	112
40	Hyetograph and Hydrographs for July 7, 1964 Rainstorm	113
41	Ten-Min Unit Hydrograph for Oakdale Basin	117
42	S-Curve and One-Min Unit Hydrograph for Oakdale Basin	118
43	Sensitivity of Computed Hydrograph to Horton's Infiltration Parameters	122-123

<u>No.</u>		<u>Page</u>
A-1	Block Diagram for Frequency Analysis Computer Program	143
A-2	Arrangement of Card Decks for Frequency Analysis Computer Program	145
A-3	An Example of Data Card Set 1 for Frequency Analysis Computer Program	149
A-4	An Example of Data Card Set 2 for Frequency Analysis Computer Program	155
A-5	An Example of Data Card Set 3 for Frequency Analysis Computer Program	155
A-6	An Example of Data Card Set 4 for Frequency Analysis Computer Program	159
A-7	Sample Printout of Frequency Analysis Computer Program	163-165
B-1	Examples of Data Card Sets 1 and 2 for Illinois Surface Runoff Model	189
B-2	Examples of Data Card Set 3 for Hyetographs for Illinois Surface Runoff Model	189
B-3	Examples of Data Card Set 4 for Subcatchments for Illinois Surface Runoff Model	195
B-4	Examples of Data Card Set 5 for Sewer Nodes for Illinois Surface Runoff Model	195
B-5	Arrangement of Input Card Decks for Illinois Surface Runoff Model	197
B-6	Layout of Conceptual Drainage Basin	198
B-7	Data Representation for Conceptual Basin	199
B-8	Sample Output from Computer Program of Illinois Surface Runoff Model	
C-1	Three Examples of Data Set 1 for Sewer Water Quality Model	218
C-2	Two Examples of Data Set 2 for Sewer Water Quality Model	218
C-3	An Example of Data Set 3 for Sewer Water Quality Model	218
C-4	Input Card Deck Arrangement for Sewer Water Quality Model	223

<u>No.</u>		<u>Page</u>
C-5	Sample Printout of Sewer Water Quality Model	224
D-1	Hypothetical Drainage Basin	232
D-2	Hyetograph for the Hypothetical Basin	234
D-3	Input Data Cards for the Example for Illinois Surface Runoff Model	236
D-4	Input Data Cards for the Example for ISS Model	237
D-5	Sample Printout for the Example	238
D-6	Runoff Hydrograph for the Example	239

TABLES

<u>No.</u>		<u>Page</u>
1	Statistics of Parameters for Summer Rainstorms at Urbana, Illinois	16
2	Conditional Statistics of Rainstorm Parameters for Different Durations	27-31
3	Dimensions of Gutters of Oakdale Avenue Drainage Basin	47
4	Dimensions of Alleys of Oakdale Avenue Drainage Basin	52
5	Dimensions of Sewers of Oakdale Avenue Drainage Basin	52
6	Urban Runoff Prediction Methods Evaluated	88-89
7	Rational Method Computation	114
8	Comparison of Urban Storm Runoff Methods	115
9	Headings for Computation of Unit Hydrograph Method	119
10	Values of Infiltration Parameters Used for Illinois Urban Storm Runoff Method Shown in Fig. 37 to 40	124
D-1	Characteristics of Subcatchments of Example Basin	233
D-2	Physical Characteristics of Gutters of Example Basin	233
D-3	Characteristics of Sewers of Example Basin	234

ACKNOWLEDGMENTS

During the period of this project, many people have contributed to the progress of the work. Many students and assistants at the University of Illinois at Urbana-Champaign have helped in various phases of data collection, card punching, data analysis, and numerous other computations. Many engineers in various federal and local government agencies, consulting firms, research institutes and universities provided indispensable help in supplying data, information exchanges, and discussions, although only selected data are used in this report and not all the ideas provided by these people are presented here. The names of these people are too numerous to be listed here and their contributions are greatly appreciated. Particular thanks are due Professor Wayne Huber of the University of Florida, Dr. Albin Brandstetter of Battelle Pacific Northwest Laboratories, Dr. P. E. Wisner of James F. MacLaren, Ltd., and Messrs. Marshall Suloway, Louis Koncza and David Westfield of the Department of Public Works, City of Chicago for their help.

Mr. A. Osman Akan, Graduate Research Assistant in the Department of Civil Engineering, University of Illinois at Urbana-Champaign, contributed most to the successfulness of this research project. He programmed the Illinois Surface Runoff Model and the Water Quality Model, provided the interface between these models and the Illinois Storm Sewer System Simulation Model, performed the tedious and painstaking testing and computer computations, participated in the field survey for data collection, and wrote the subsection on Program Description for the Illinois Surface Runoff Model and Appendices B and C. Without his help, this report would not be materialized. Mr. T. A. Ula, Graduate Research Assistant, performed the frequency analysis and wrote Appendix A in addition to his contributions in data collection and various aspects of computations.

Throughout this research project, Mr. Chi-Yuan Fan, Project Officer, and Mr. Richard Field, Chief, Storm and Combined Sewer Section, EPA Municipal Environmental Research Laboratory, provided smooth and fruitful cooperation. Their help in supplying information on data sources and coordination with other researches is indispensable and deeply appreciated.

Thanks are also due Mrs. Norma Barton for her typing of the report and other materials for the project.

SECTION I

SUMMARY AND CONCLUSIONS

The purpose of this report is to provide information which is useful for the operation and management of urban storm and combined sewer runoff and also useful for the design and operation of overflow treatment and other control facilities. This report is divided into three major parts. The first part contained in Section IV is precipitation analysis on depth-duration-frequency analysis of short return period (high frequency) rainstorms. Conditional probability is utilized and method of application on urban runoff problems is demonstrated using the hourly precipitation record available from the U.S. Environmental Data Service, National Climatic Center.

The second part of the report consists of Sections VI, VII, and VIII describing the Illinois Urban Storm Runoff method. It includes the development of the Illinois Urban Surface Runoff model to couple with the Illinois Storm Sewer System Simulation Model and the formulation of a non-reactive water quality model to compute the concentration of the pollutants of urban runoff. In the surface runoff model, Horton's formula is used to evaluate infiltration. The overland flow is computed by using the kinematic wave method together with Darcy-Weisbach's formula to estimate the friction slope. The gutter flow is computed by using the kinematic wave method together with Manning's formula to estimate the friction slope. Inlets to catch basins are classified in types according to their geometry, and weir and orifice formulas are used to estimate the discharge. The sewer flow routing utilizes the complete St. Venant equations accounting for the junction backwater effects. An example to illustrate the application of the Illinois Urban Storm Runoff model on a hypothetical drainage basin is given in Appendix D.

The third part of the report consisting of Sections V, IX, and X is a comparative evaluation of eight selected urban runoff prediction methods using the data from the Oakdale Avenue Drainage Basin in Chicago. The methods evaluated are the rational method, unit hydrograph method, Chicago hydrograph method, British Transport and Road Research Laboratory method, University of Cincinnati Urban Runoff method, Dorsch Hydrograph Volume Method, EPA Storm Water Management Model, and the Illinois Urban Storm Runoff method. This part of study is believed to be of particular interest to practicing engineers. The evaluation is made by using the recorded hyetographs of four rainstorms applying the methods to compute the predicted runoff hydrographs and then compare the results with the recorded hydrographs. This comparative study suggests that the most suitable method to be used for an urban runoff problem depends on the objective and accuracy required for the project. If only a quick simple approximate result of peak runoff rate is needed, the rational method is often satisfactory, whereas for a project involving high accuracy and details of the temporal and spatial distribution of the runoff are required, the Illinois Urban Storm Runoff method will be a

suitable choice, and the EPA Storm Water Management Model may be the alternative if the downstream backwater effects are not important. For in-between accuracy, the unit hydrograph method is recommended, if possible. When the unit hydrograph for the drainage area is not available, in most cases the Transport and Road Research Laboratory method appears to be superior to the Chicago and University of Cincinnati methods.

SECTION II

RECOMMENDATIONS

Based on the results of this investigation, the following recommendations related to the determination of flow rate of urban stormwater runoff are made:

1. The most suitable method to be used for determination of urban storm runoff depends on the objective and size of the project, the required accuracy and detail of the flow rate determination, and the available data.
2. For the purpose of stormwater runoff control and management, when required results are extensive (such as discharge and depth or velocity at different times and at many locations of the drainage basin) and the required accuracy is high such as in the case of a high-valued urban area or a high-cost project, and the detailed basin data is available, the Illinois Urban Storm Runoff method appears to be a suitable choice. If the downstream backwater effect of sewer junctions is not important, the EPA Storm Water Management Model is a good alternative; whereas the Dorsch Hydrograph-Volume Method may be the next choice.
3. For a cheap, simple and quick estimation of the peak runoff rate without requiring the entire runoff hydrograph, the rational method can often be used satisfactorily. However, one should always bear in mind the limited accuracy the rational method can provide -- a sacrifice of accuracy for the sake of simplicity.
4. For projects or problems requiring determination of the entire or part of the runoff hydrograph with moderate accuracy, the unit hydrograph method is the simplest and cheapest to use if the unit hydrograph is available or can be reliably derived. In using the unit hydrograph, one should check that there should be no significant change in basin characteristics. If the unit hydrograph is not available, the order of choice would be the British Transport and Road Research Laboratory method, Chicago hydrograph method, and University of Cincinnati Urban Runoff Model.
5. The comparative study is made on only eight methods and is neither exhaustive nor exclusive. Other methods not mentioned here may also be useful.

6. Existing and available data on urban rainfall, runoff, and basin characteristics are generally inadequate for a reliable accurate evaluation of the methods. Neither are the data adequate for engineering operation, management and design purposes. This inadequacy is in both details and accuracy. For example, the best readily available rainfall data, the hourly data from the Environmental Data Service, National Climatic Center, is inadequate in view of many urban drainage basins having a time of travel much shorter than an hour.
7. More study on urban infiltration would be desirable. This includes the determination and listing of infiltration parameters such as f_c and k in Horton's formula or similar parameters in Philip's formula for typical urban surface conditions, and data on such information as antecedent moisture condition related to infiltration.
8. A detailed accurate evaluation of the surface runoff part of the methods is desirable. Because of the lack of sufficient detailed geometric (percentage and distribution of roofs, lawns, sidewalks, driveways, etc.) and infiltration data, it is believed that the effects of the surface runoff on the methods have not been adequately evaluated. In addition, surface runoff hydrographs at certain urban locations will also be useful for engineering purposes such as street drainage design.
9. Further study on hydraulic characteristics of inlets is necessary. There is no use to have a highly sophisticated surface runoff model if its downstream control, the inlet, cannot be accurately modeled. Existing information is inadequate to represent the large number of types of inlets now being used. From both hydraulic and economic viewpoints, standardization of the types and geometry of the inlets is desirable.
10. Improvement on the manner to handle surcharge and supercritical flow in sewers would be useful and desirable.
11. Further study on the differences between design and flow prediction purposes on simulation modeling and data requirements may be useful for urban storm runoff management.
12. In view of the probabilistic nature of the physical conditions of the drainage system, such as clogging of inlets and gutters, interference of parked vehicles on street and gutter flows, change of roughness of sewers, etc., development of a probabilistic method to account for such uncertainties will be useful for both design and operation purposes.

13. It is of course possible to further improve the hydraulic aspects of the existing sophisticated methods. For instance, the kinematic wave routing of the overland and gutter flows can be replaced by solving the more accurate St. Venant equations. However, at present such improvement appears to be unfruitful and immature because of the uncertainties involved in the basin physical properties and the detail and reliability of the data. Furthermore, such improvement would require considerably more computer time, thus making the method impractical.
14. An advanced stochastic approach to analysis and predicted high frequency rainfall as an alternative to the method proposed in Sec. IV-1 may be useful.

SECTION III

INTRODUCTION

III-1. PROBLEMS OF URBAN STORM WATER RUNOFF

Metropolitan areas, in the United States as well as elsewhere in the world, are growing at an unprecedented rate. One result often associated with urbanization is the deterioration of the living environment due to either lack of comprehensive planning or incapability of the cities to keep pace with the growth. Among the vital facilities in preserving the living environment, urban sewer drainage systems affect directly the quantity and quality in disposing urban waste water. Large amounts of money and resources are involved in the design, construction, modification, operation and maintenance of urban sewer systems. There are two major types of problems related to draining urban storm water: (1) flooding due to inadequate sewer capacity causing damage of properties and disruption of traffic and other human activities; and (2) pollution due to storm runoff. The flooding problem is a design problem involving design rainstorms having return period of once in several years. Many methods have been developed for sewer design purpose in the past 130 years (Chow, 1962, 1964). The common objective of these methods is to provide a design flow for sizing the sewers of a new storm drainage system or modification of an existing system with adequate capacities to dispose of this once-in-many-year design flow without flooding.

Conversely, the storm runoff pollution problem is an operation and management problem. It involves consideration of rainstorms with frequencies of several times in a year. For an existing drainage system the problem involves the management of the time distribution of the quantity and quality of storm water so that the runoff would not overload the treatment facilities or unacceptably pollute the receiving water bodies. This may require some modification of the drainage facilities. For new drainage systems a balanced design considering both the pollution control and adequate capacity to avoid flooding would require an optimization analysis.

During the past decade the public as well as the engineering profession has been greatly alarmed by the pollution aspects of urban storm runoff. Many investigations have been sponsored by EPA and by other agencies on the extent of urban runoff pollution (e.g., see American Public Works Association, 1967, 1969; Engineering-Science, Inc., 1967; Envirogenics Company, 1971; Hawkins and Judd, 1972; Sartor and Boyd, 1972; Weibel et al., 1964). Recent studies on urban storm runoff pollution and efforts on its control and management have been summarized in two excellent literature reviews (Field and Weigel, 1973; Field and Szeley, 1974).

One fundamental prerequisite for an efficient and economic design and operation of urban sewer systems is a reliable method to predict the quantity and quality of water handled by the system, particularly the

time distribution of runoff due to rainfall at important locations such as junctions and overflow facilities. Numerous methods have been proposed to estimate rainstorm runoff. Many of these methods are of regional nature whereas many others are applicable only to rural or similar natural drainage basins. For those methods applicable to urban areas, some treat the drainage area as a "black box" without considering the time or space distribution of the runoff in the drainage system; others treat the drainage system as sequential overland and channel flows without considering the detention storage in the drainage system due to backwater effects of the junctions. Furthermore, many of the improved methods which have been proposed recently are not yet widely accepted mainly because their relative merits have not been assessed, nor have they been compared in a comprehensive manner on a common basis to the previously developed methods.

however, in order to comply with recent pollution control laws, attempts have been made to utilize the storage capacity of a sewer system to detain storm runoff and to control overflow in order to reduce the cost in handling urban waste water problems (Poertner, 1972; Anderson et al., 1972; Field and Struzeski, 1972). Examples of such attempts are the automatic control systems built or proposed in Minneapolis-St. Paul (Anderson, 1970; Tucker, 1971), Seattle, San Francisco, and Detroit (Field and Struzeski, 1972). Obviously, a reliable storm runoff prediction method would be particularly useful and beneficial for such efforts. In fact, without a reliable runoff quantity prediction it is most unlikely that a satisfactory runoff quality prediction can be achieved.

III-2. OBJECTIVES AND SCOPE OF STUDY

As discussed in the preceding subsection, a "demonstration" type investigation to evaluate quantitatively on a common basis the relative merits of the conventional as well as recently developed runoff prediction methods is needed for improvement in urban storm water management and pollution control. Since many of the urban storm runoff models are being compared quantitatively in an EPA project at Battelle Pacific Northwest Laboratories using hypothetical data, it is not the intention of this investigation to evaluate all of the existing urban storm runoff models. Accordingly, the major objectives of the present study are: (a) to develop a method of rainfall depth-duration-frequency analysis suitable for urban storm runoff management; (b) to develop a surface runoff model for determination of stormwater quantity and flow rate; and (c) to evaluate this new method using actual field data and also compare the results quantitatively with other relatively popular methods using the same field data from the Oakdale Avenue Drainage Basin in Chicago. The methods compared in this study include the rational method, unit hydrograph method, Chicago hydrograph method, British Transport and Road Research Laboratory (TRRL) method, University of Cincinnati method, Dorsch Hydrograph-Volume-method, EPA Storm Water Management Model (SWMM), and the Illinois Urban Storm Runoff method. The description of the last

method is given in Sections VI and VII. Brief descriptions of the other seven methods are given in Section IX.

This report is written in such a manner that different parts can be read independently for the convenience of the engineers having different interests. For example, for those engineers who are interested only in adopting the computer programs of the Illinois Urban Storm Runoff method for engineering problems but not in its development nor its comparison with other methods, it is adequate to read only Sec. VI-4, VII-3, VIII-2, and Appendices B, C, and D. For those who are only interested in applying the method on rainfall depth-duration-frequency analysis for urban storm runoff problems it is adequate to read only Sec. IV-1 and Appendix A. For those who are interested only in the quantity aspects of storm runoff, Section VIII and Appendix C can be omitted.

SECTION IV

PRECIPITATION ANALYSIS

Not all the rainwater falling on an urban area becomes runoff. There are infiltration and other losses called abstractions. The total precipitation after deducting the abstractions is called precipitation excess. Since both precipitation and abstractions are statistical quantities, precipitation excess is also of statistical nature. In this section precipitation and abstractions are discussed in view of urban storm runoff.

IV-1. RAINFALL DEPTH-DURATION-FREQUENCY ANALYSIS FOR URBAN RUNOFF

Natural rainfalls have finite duration and areal and temporal variation of their intensities. It has yet to be found that two rainfalls are identical. Thus rainfall data is analyzed statistically to be useful for engineering purposes. The average rainfall intensity of a rainstorm is a function of its duration, frequency, and area, which has been discussed extensively elsewhere (e.g., U.S. National Weather Service, 1961; Chow, 1964). There are two types of rainfall information needed for urban storm runoff management purposes. The first is the duration and maximum intensity for rainfalls having long return periods of a number of years to be used for design and safety considerations. The other is the information on high frequency rainstorms with return periods less than a year to be used for operation and pollution control purposes.

Because of the large number of rainfalls involved for a given location, conventionally in engineering hydrology as well as in meteorology only the maximum values in the form of partial duration series or annual maximum series are analyzed to establish the rainfall intensity-duration-frequency relationship for long return period events. This is done so because usually for the purpose of design of drainage facilities based on catastrophic failure concept (Yen and Ang, 1971) rainfall of small return period is usually of no significance. However, this is not the case from an operational viewpoint for the control of pollution due to urban storm runoff for which every rainstorm contributes to the problem. Because of the lack of dilution effect due to their small volume, the low return period rainstorms probably contribute considerably more to pollution per unit volume of water than the long return period ones. Treatment plants, detention storages, overflows and other sewer facilities designed for small capacities corresponding to more frequent rainfalls would not be able to handle the runoff from less frequent rainstorms. Contrarily, such facilities designed to operate at full capacity only once in every several years would be costly and unjustified if the safety consideration does not require it.

Obviously, the volume and time distribution of storm runoff quantity and quality from an urban drainage system depend on the areal and temporal distributions of the rain falling on the urban basin. Consequently, the depth, duration, and frequency of the rainfall as well as other parameters defining the internal pattern of the rainstorm should all be considered. In fact, the time elapsed between successive rainfalls is also an important factor in determining the quantity and especially the quality of storm runoff. Conceivably, most of the water from a rainstorm following soon after an earlier heavy rainfall would become runoff and the quality of the water would be relatively better.

Information on average rainfall intensity for a given duration for long return period rainstorms useful for urban runoff studies have been well established and can readily be found (e.g., Chow, 1953; U.S. National Weather Service, 1961). Unfortunately, analytical information on high frequency rainstorms with return periods less than a year which is useful for urban engineering purposes is practically nonexistent. This lack of information is due mainly to the large amount of data needed to be analyzed and to the difficulty in defining precisely the duration and intensity of a rainstorm. The purpose of this part of the study is to provide a practical model on a probabilistic basis for rainfall input for urban storm runoff studies and operations.

The frequency analysis methods for rainfalls of long return period can be extended to short return period rainfalls. However, unlike the long return period case for which data can be selected to form a partial series for the analysis, for short return period analysis the entire set of data, i.e., all the rainstorms recorded, is utilized. Grayman and Eagleson (1969) applied this concept on hourly rainfall data for 546 rainstorms in a 5-year period at Boston.

Ideally, the precipitation data used for short return period frequency analysis should be a continuous record of hyetograph (rainfall-time curve). For most precipitation recording-gaging stations in the United States, the most detailed precipitation data readily available from the Environmental Data Service, National Climatic Center* are hourly records. Since many urban drainage basins have the time of travel of surface runoff less than an hour, the hourly rainfall data is obviously inadequate and unsatisfactory. Data of 5- or 10-min intervals would be much more satisfactory. Unfortunately, rainfall records having time intervals shorter than one hour may only be obtained by the user from the original record, if available, which is a very time consuming process and most unlikely to be undertaken by practicing engineers. Therefore, from a

*Hourly rainfall data for recording raingages in the U.S. National Weather Service system are available at cost on punched cards from U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Environmental Data Service, National Climatic Center, Federal Building, Asheville, N.C. 28801. Under special circumstance 15-min rainfall data may also be available.

practical viewpoint, this part of research on rainfall frequency analysis method is developed by using hourly rainfall records. Nevertheless, the methodology is equally applicable to data of other time intervals.

The method of analysis which has been written in a computer program (Appendix A) will be described in the following. The data used to demonstrate the application of the method are the point hourly precipitation data on punched cards as provided by the National Climatic Center for the Morrow Plot raingage at Urbana, Illinois, (11-8740 Urbana), covering 14 years from 1959 to 1972. Because of the seasonal characteristics of rainstorms, the analysis is carried out separately for different seasons. The analysis for the record of June, July, and August consisting of 455 rainstorms is presented here as an example.

(A) Identification of Rainstorms

A rainstorm is defined here as a period of continuous non-zero rainfall. Since the input data provided on punched cards do not identify traces (rainfall less than 0.01 in./hr or 0.25 mm/hr), there is no way to differentiate traces from hours of no rainfall in the data. The duration of a rainstorm, t_d in hours, is defined as the length of time between the beginning and the end of a continuous non-zero rainfall. The total volume of a rainstorm as customarily expressed in depth, D in in. or mm covering the entire area considered, is equal to the sum of the depth for each time interval within the duration of the rainstorm, i.e.,

$$D = \sum_{j=1}^n d_j \quad (1)$$

in which d_j is the depth for the j -th time interval and n is the number of time intervals of the rainstorm. In engineering practice, as a matter of convenience, equal time interval Δt is usually used and the standard time interval used here is one hour according to the standard U.S. National Weather Service data, although other time intervals can also be specified. The average intensity of the rainstorm, i , is defined as equal to D/t_d , and usually expressed in in./hr or mm/hr. The elapse time between successive rainstorms, t_b , is defined as the time between the end of a rainstorm and the beginning of the next rainstorm. The computer program listed in Appendix A traces the entire record, identifies the rainstorms and determines the values of D , i , t_d , and t_b .

(B) Calculation of Rainstorm Parameters

A schematic drawing of an example hyetograph is shown in Fig. 1. The other parameters of the rainstorms essential for a statistical analysis are computed as follows. The standard deviation of the rainstorm depth, σ_d in in. or mm, is computed as

$$\sigma_d = \left[\frac{\sum_{j=1}^n (d_j - \bar{d})^2}{n} \right]^{1/2} \quad (2)$$

where the average depth per time interval, $\bar{d}$ in in. or mm, is

$$\bar{d} = \frac{\sum_{j=1}^n d_j}{n} \quad (3)$$

The first moment arm of the hyetograph with respect to the beginning time of the rainstorm, $\bar{t}$ in hr, is

$$\bar{t} = \Delta t \left[\frac{\sum_{j=1}^n (j - 0.5) d_j}{\sum_{j=1}^n d_j} \right] \quad (4)$$

and the corresponding second moment arm, G in hr^2 , is

$$G = (\Delta t)^2 \left[\frac{\sum_{j=1}^n (j - 0.5)^2 d_j}{\sum_{j=1}^n d_j} + \frac{1}{12} \frac{\sum_{j=1}^n d_j}{\sum_{j=1}^n d_j} \right] \quad (5)$$

(C) Nondimensional Hyetograph

In order to describe in more general terms the time distribution of the rainfall of a rainstorm, the hyetograph is nondimensionalized by using the rainstorm depth D and duration t_d as the nondimensionalizing parameters. Therefore, for a time interval, the nondimensional depth $d_j^o = d_j/D$ where the superscript (o) represents the nondimensional quantity. Accordingly, $D^o = \sum_{j=1}^n d_j^o = 1$ and $t_d^o = 1$. The nondimensional average intensity $i^o = D^o/t_d^o = 1$. Similarly, the nondimensional average depth is

$$\bar{d}^o = \frac{\sum_{j=1}^n d_j^o}{n} = \frac{1}{n} \quad (6)$$

The nondimensional standard deviation of the depth is

$$\sigma_d^o = \left[\frac{\sum_{j=1}^n (d_j^o - \bar{d}^o)^2}{n} \right]^{1/2} = \frac{\sigma_d}{D} \quad (7)$$

The first moment arm of the nondimensional hyetograph is

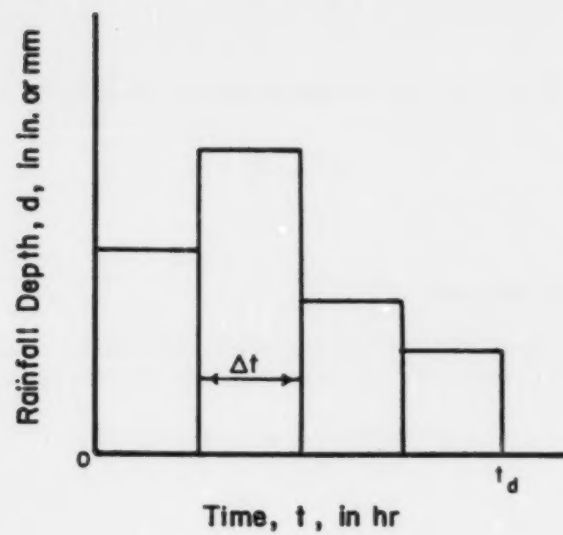


Fig. 1. Example hyetograph

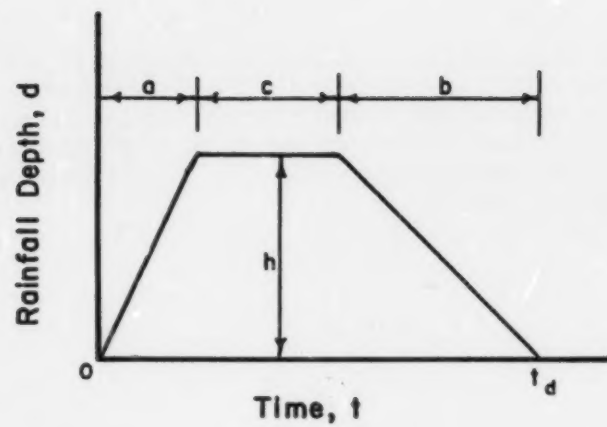


Fig. 2. Trapezoidal representation of hyetograph

$$\bar{t}^0 = \frac{\bar{t}}{t_d} \quad (8)$$

The second moment arm of the nondimensional hyetograph is

$$G^0 = \frac{G}{t_d^2} \quad (9)$$

(D) Shape of Hyetographs

The shape of the hyetographs may be approximated by some simple geometric figures. Assuming that the hyetographs can be represented by trapezoids shown in Fig. 2,

$$t_d = a + b + c \quad (10)$$

$$\bar{d} = \frac{h}{2} \left(1 + \frac{c}{t_d} \right) \quad (11)$$

$$\bar{t} = \frac{t_d(t_d + a + c) + c(2a + c)}{3(t_d + c)} \quad (12)$$

and

$$G = \frac{t_d^3 + (a + c)t_d^2 + (a + c)^2(t_d + c) + ca(2a + c)}{6(t_d + c)} \quad (13)$$

Equations 10 through 13 can be solved simultaneously for a , b , c , and h so that the trapezoid representing the hyetograph can be determined. To demonstrate the methodology in application, it is assumed that a special case of the trapezoidal shape with $c = 0$, i.e. triangles, can be used to approximate the Urbana rainfall data. For this triangular case, solving Eqs. 10, 11, and 12 yields

$$a = 3\bar{t} - t_d \quad (14)$$

$$b = 2t_d - 3\bar{t} \quad (15)$$

and

$$h = 2\bar{d} \quad (16)$$

For nondimensional hyetograph,

$$a^0 = a/t_d = 3\bar{t}^0 - 1 \quad (17)$$

$$b^0 = b/t_d = 2 - 3\bar{t}^0 \quad (18)$$

and

$$h^0 = h/D = 2\bar{d}/D = 2/n \quad (19)$$

The statistics of the parameters for the 455 summer rainstorms at Urbana, Illinois are given in Table 1.

(E) Frequency Analysis of Rainstorm Parameters

With the rainstorm parameters computed for every rainstorm in the record, a one-way frequency analysis can subsequently be made for each parameter. The computer program (Appendix A) has a one-way frequency analysis subroutine which tabulates for any given parameter the frequencies (number of observations over given intervals), relative frequencies (frequency divided by the total number of observations), probability densities (relative frequency divided by the interval size) and non-exceedance probabilities (cumulative relative frequencies). The mean and standard deviation of the parameter are also calculated and the maximum and minimum values are found (Table 1). Histograms of the probability densities for the rainstorm parameters can then be plotted as shown in Figs. 3, 4, 5, and 6 for the rainstorm depth, duration, intensity, and elapse time between rainstorms, respectively.

(F) Fitting of Probability Density Functions

The next step is to fit some probability density functions to the histograms of the rainstorm parameters. Exponential distribution and gamma distribution have a non-negative range and have been applied in the analysis of non-negative valued rainstorm parameters. The probability density function for the exponential distribution is

$$f(x) = \frac{1}{B} e^{-x/B} \quad x \geq 0 \text{ and } B > 0 \quad (20)$$

where both the expected value and the standard deviation of the distribution are given by B. This single-parameter distribution can easily be fitted to data (Grayman and Eagleson, 1969) but the fact that it has the same expected value and standard deviation makes its use difficult to justify in most applications. The exponential density functions fitted in Figs. 3 through 6 all assume a value of B equal to the observed mean. Use of the mean rather than standard deviation for B implicitly implies that the mean is more "meaningful" than the standard deviation. The probability density function for the gamma distribution is

$$f(x) = \frac{1}{\Gamma(C)B^C} x^{C-1} e^{-x/B} \quad x \geq 0 \text{ and } B, C > 0 \quad (21)$$

Table 1. STATISTICS OF PARAMETERS FOR SUMMER
RAINSTORMS AT URBANA, ILLINOIS

Parameter	Mean	Standard Deviation	Min	Max
t_b , hr	59.4	81.9	1	744
t_d , hr	2.45	2.08	1	14
D , in.	0.31	0.51	0.01	3.52
D , mm	7.87	13.0	0.25	89.4
i , in./hr	0.11	0.15	0.01	1.05
i , mm/hr	2.79	3.81	0.25	26.7
σ_d , in.	0.06	0.11	0	0.87
σ_d , mm	1.52	2.79	0	22.1
$\bar{t}$, hr	1.14	1.01	0.50	7.68
G , hr ²	2.90	7.10	0.33	71.2
a , hr	0.95	1.39	-5.00	10.0
b , hr	1.50	1.80	-2.10	13.8
h , in.	0.21	0.30	0.02	2.09
h , mm	5.33	7.62	0.51	53.1
σ_d^o	0.13	0.15	0	0.48
$\bar{t}^o$	0.47	0.11	0.13	0.80
G^o	0.30	0.11	0.05	0.68
a^o	0.42	0.32	-0.62	1.40
b^o	0.58	0.32	-0.40	1.62
h^o	1.23	0.65	0.14	2.00

Number of Rainstorms = 455

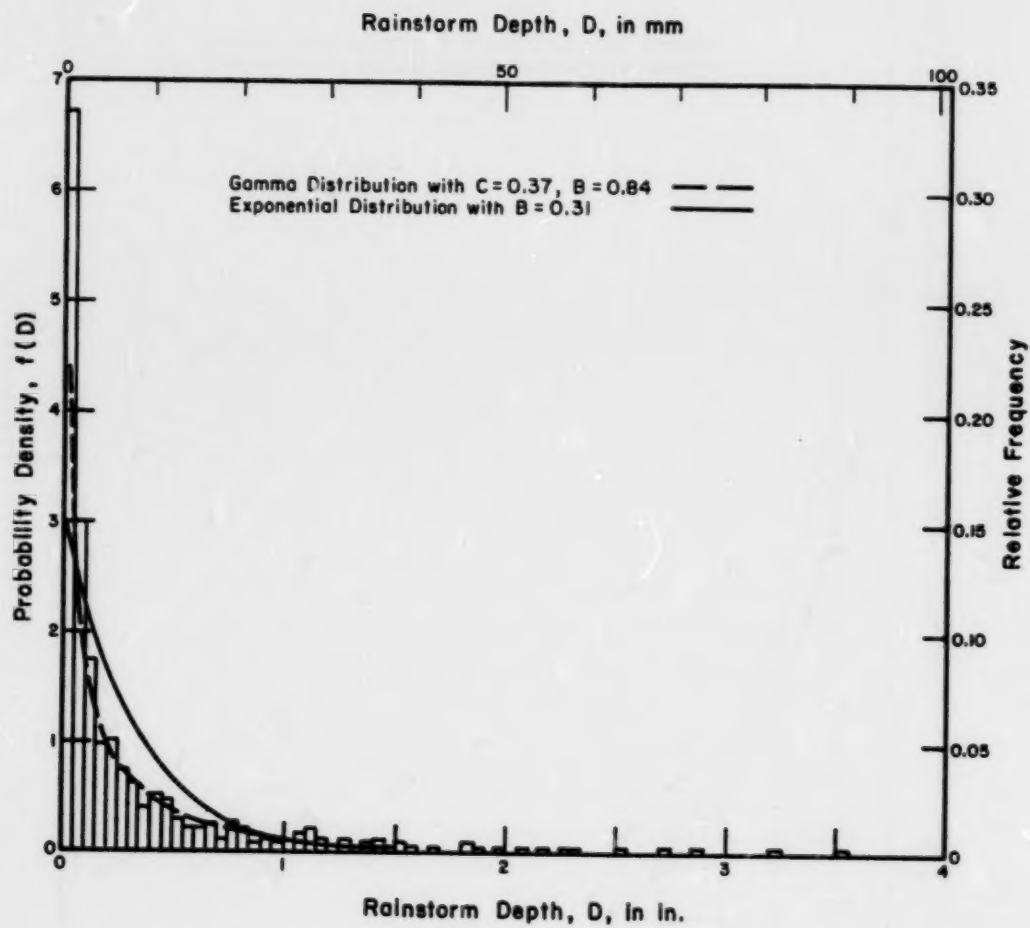


Fig. 3. Probability distribution of rainstorm depth

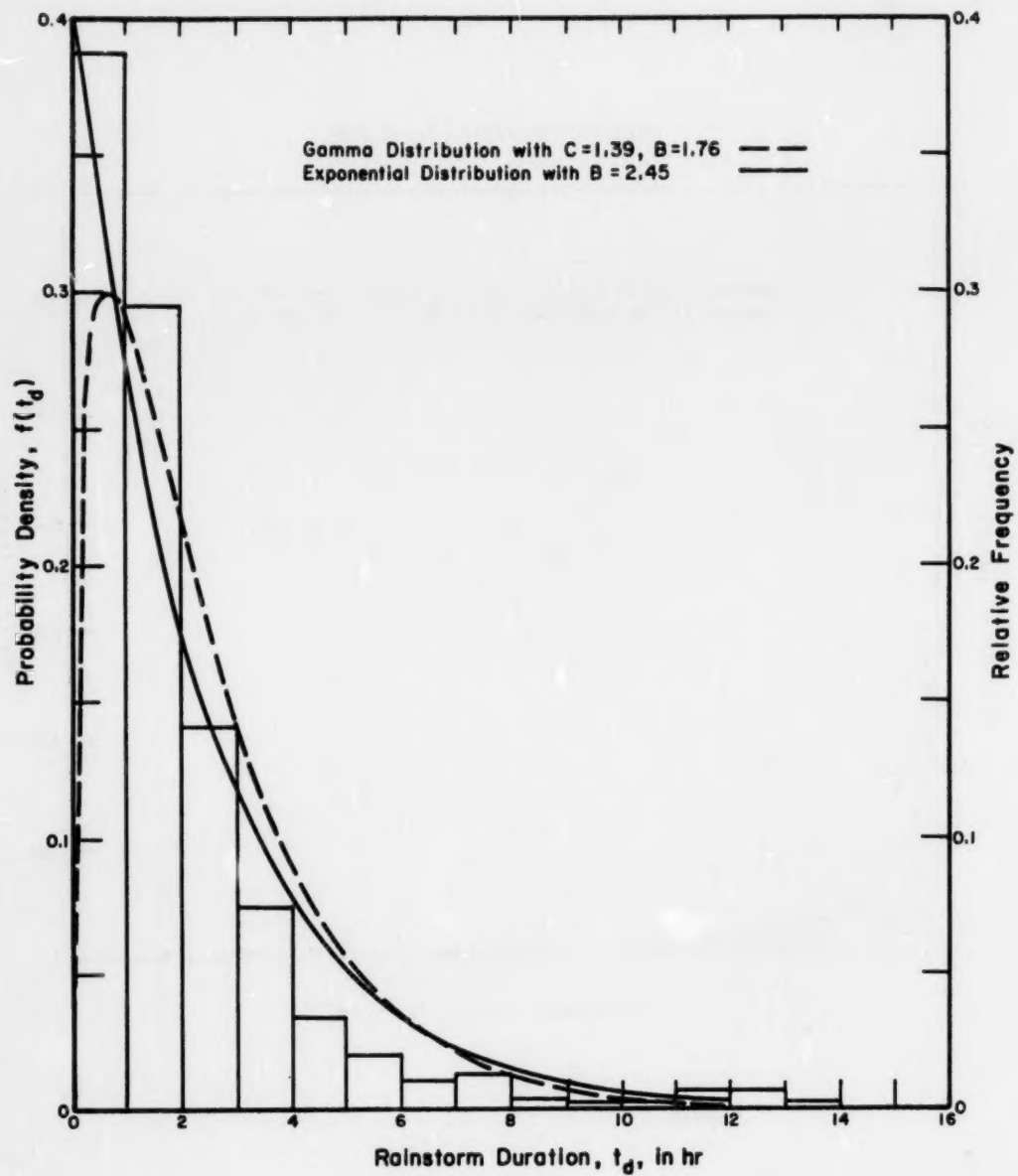


Fig. 4. Probability distribution of rainstorm duration

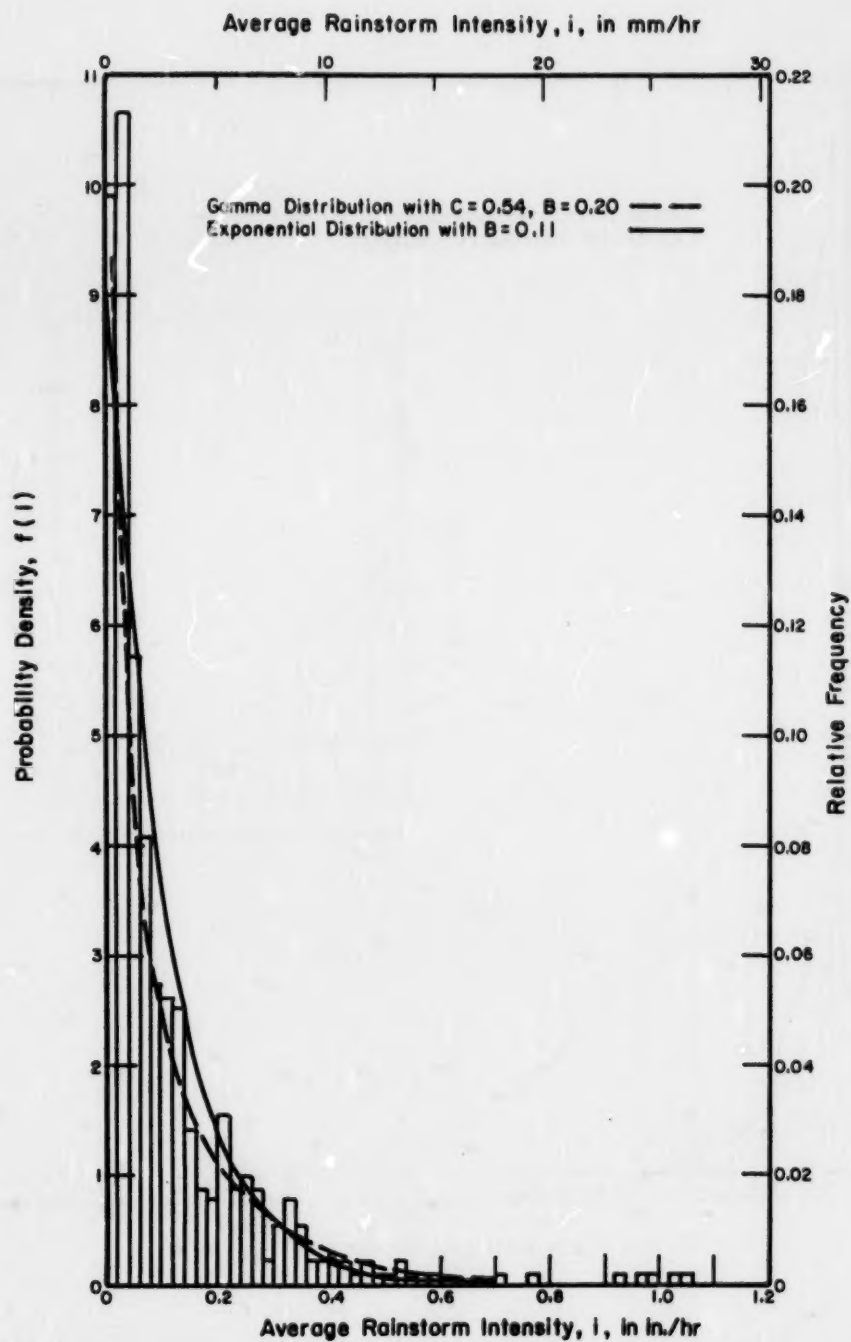


Fig. 5. Probability distribution of average rainstorm intensity

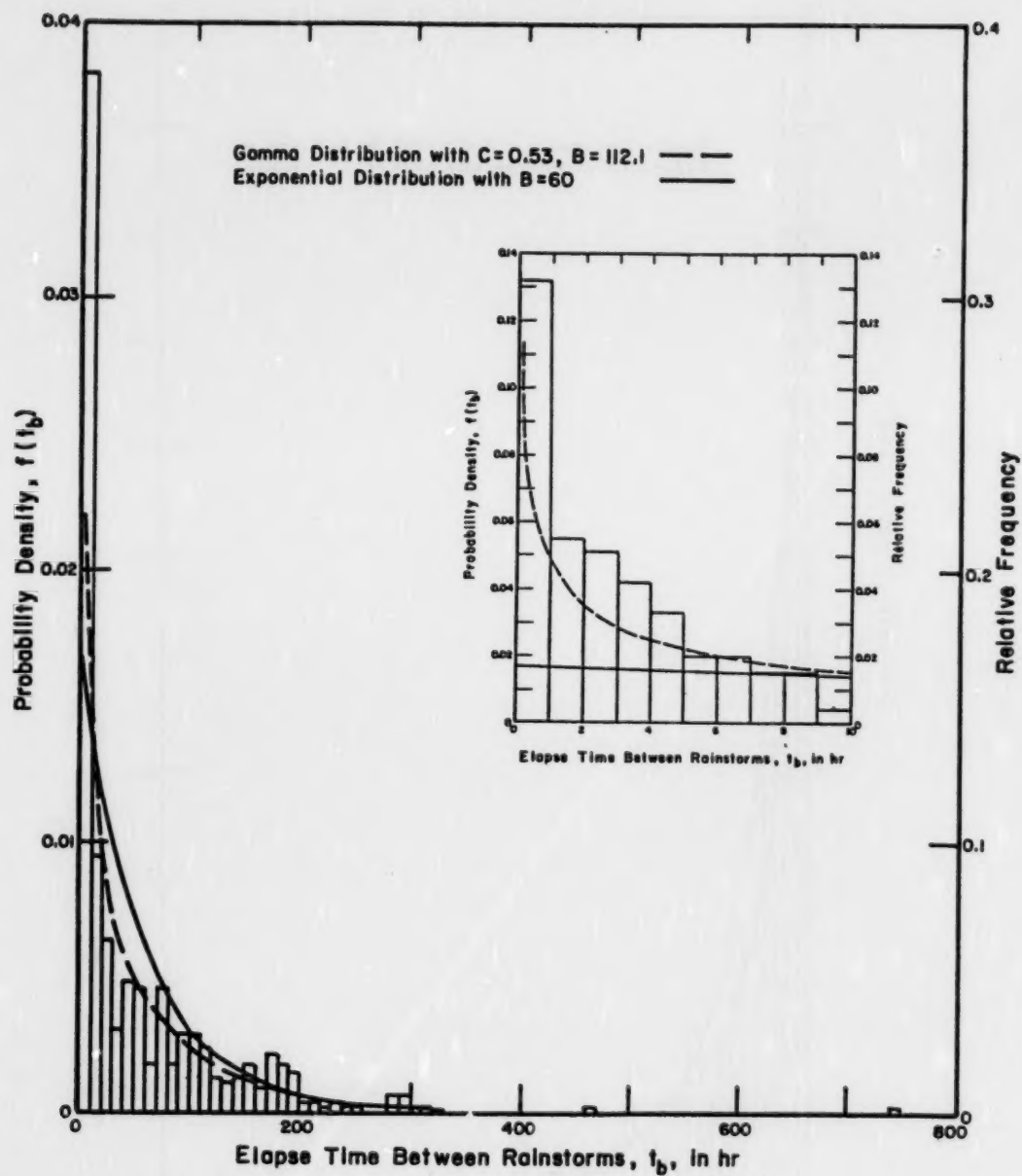


Fig. 6. Probability distribution of elapse time between rainstorms

where the gamma function

$$\Gamma(C) = \int_0^{\infty} z^{C-1} e^{-z} dz \quad \text{for all } C > 0 \quad (22)$$

The expected value

$$E(x) = CB \quad (23)$$

and the variance

$$V(x) = CB^2 \quad (24)$$

The exponential distribution is actually a special case of gamma distribution with $C = 1$. The gamma distribution being a two-parameter distribution provides an extra degree of freedom compared to the exponential distribution in fitting data, and both mean and standard deviation of the observed data can be preserved by adjusting its parameters C and B . In Figs. 3 through 6, the values of C and B for the fitted gamma probability density function are computed by using the observed values of mean and standard deviation:

$$C = E^2(x)/V(x) \quad (25)$$

$$B = E(x)/C \quad (26)$$

It can be seen from Figs. 3 through 6 that in general the gamma distribution provides a better fit to the data than the exponential distribution.

(G) Conditional Frequency Analysis of Rainstorm Parameters

As the rainstorm parameters are not truly independent, conditional probabilistic analysis is included to provide information useful for solving urban storm runoff problems. The computer program has a two-way frequency analysis subroutine which tabulates, for any two given parameters, a two-dimensional table of frequencies, relative frequencies, and probability densities. The results can then be used to plot three-dimensional histograms of joint probability densities for pairs of rainstorm parameters. However, it is a formidable task to fit bivariate joint density functions to three-dimensional histograms. Mainly for this reason and also from a practical viewpoint, in this study joint distributions are dealt with through the use of conditional distributions. The computer program has a sorting subroutine which can sort the values of a rainstorm parameter in an ascending order and can rearrange simultaneously the corresponding values of another parameter. By using this subroutine together with the one-way frequency analysis subroutine, conditional frequency analysis can be carried out for pairs of rainstorm parameters.

The example described here is to find the conditional distributions of average rainstorm intensity, i , for different rainstorm durations. The same procedure can be applied to any other pair of dependent rainstorm parameters. By using the sorting subroutine, values of t_d are sorted in an ascending order (here they vary from 1 to 14 hr), and corresponding values of i are rearranged simultaneously. For a given value or a given range of values of t_d , the corresponding i values are picked up for a one-way frequency analysis as described in (E). By repeating this one-way frequency analysis of i for different values of t_d , a set of histograms of conditional probability densities of i are obtained as shown in Figs. 7a to 7e. Exponential and gamma density functions are then fitted to these histograms with parameters evaluated based on the observed conditional means and standard deviations. Tables 2a to 7e give the values of conditional mean, standard deviation, minimum and maximum of each rainstorm parameter for different values of t_d .

The above procedure gives a set of fitted conditional density functions for i corresponding to different values of t_d . Although it is not necessary from a practical viewpoint, it is often convenient for easy application to obtain a single expression as the conditional density function of i given t_d ; i.e. $f(i|t_d)$. For the present example, this is done by using the exponential and gamma density functions and by expressing the parameters of these density functions as functions of the storm duration, using the values fitted in Figs. 7a to 7e. For the gamma distribution, as shown in Fig. 8, assuming an exponential relationship between B or C and t_d , the fitted expressions are

$$B = 0.33t_d^{-0.79} \quad (27)$$

and

$$C = 0.31t_d \quad (28)$$

Thus, from Eqs. 21, 27 and 28, the conditional gamma density function is

$$f(i|t_d) = \frac{1}{\Gamma(0.31t_d)} (3t_d^{0.79})^{0.31t_d} i^{0.31t_d-1} \exp(-3it_d^{0.79}) \quad (29)$$

Likewise, for the exponential distribution, a plot of B against t_d (Fig. 9) yields

$$B = 0.08t_d^{0.37} \quad (30)$$

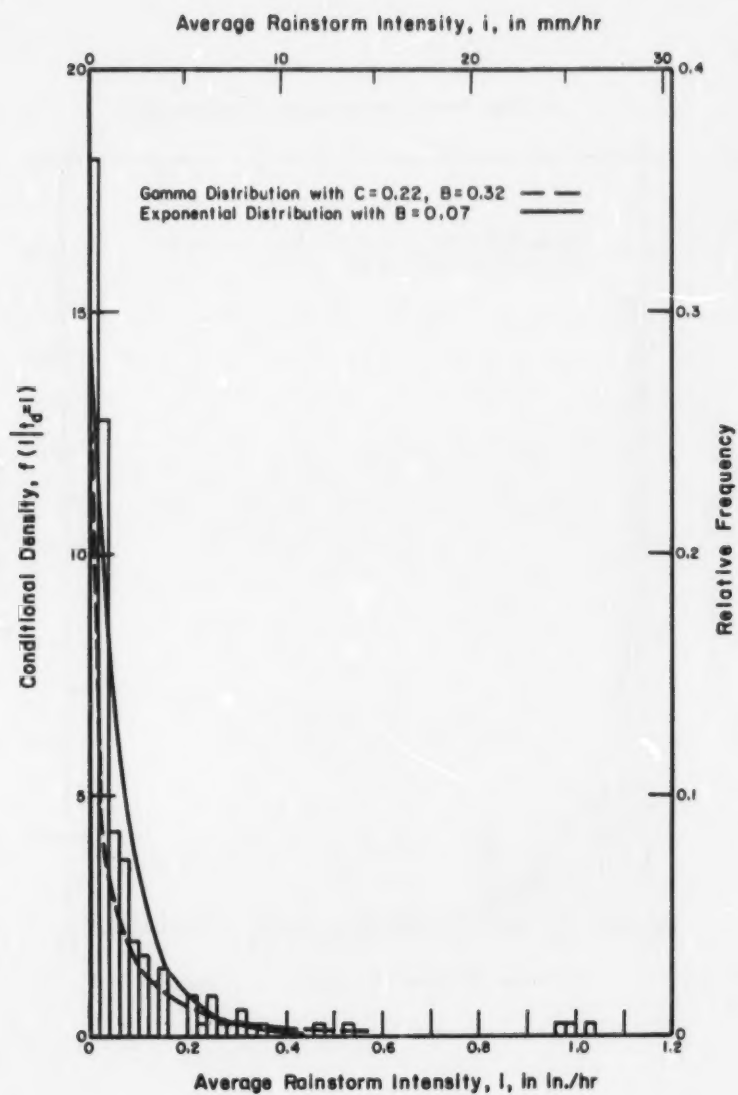


Fig. 7a. Conditional distributions of average rainstorm intensity,
 $t_d = 1$ hr

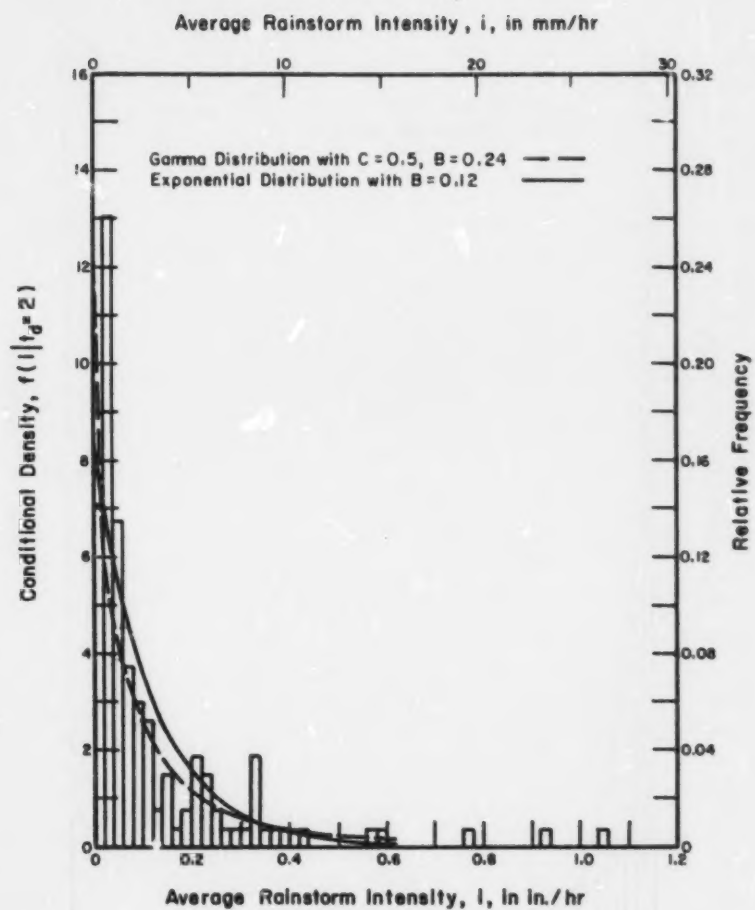


Fig. 7b. Conditional distributions of average rainstorm intensity,
 $t_d = 2$ hr

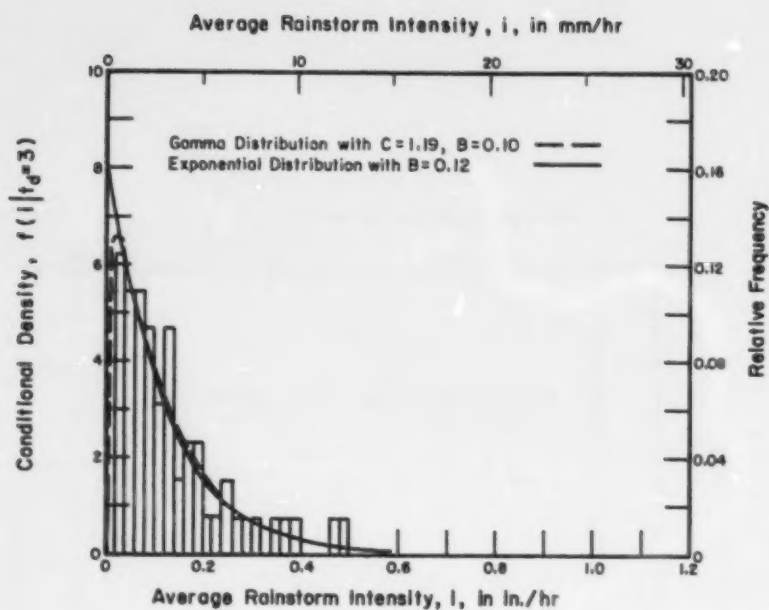


Fig. 7c. Conditional distributions of average rainstorm intensity,
 $t_d = 3$ hr

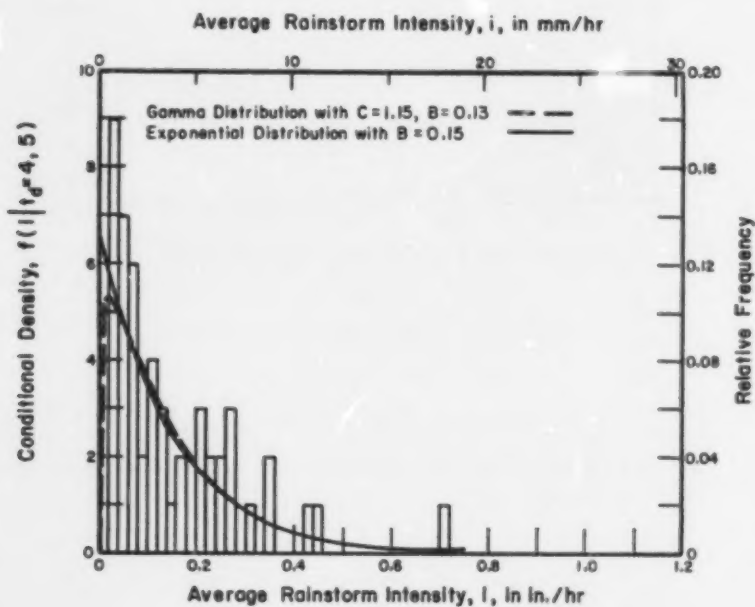


Fig. 7d. Conditional distributions of average rainstorm intensity,
 $t_d = 4, 5$ hr

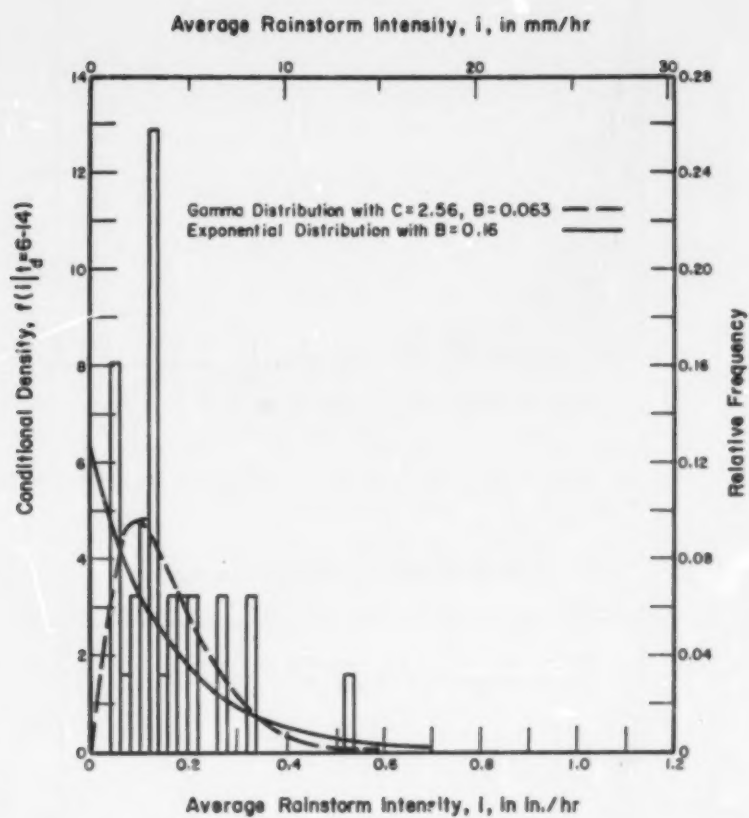


Fig. 7e. Conditional distributions of average rainstorm intensity,
 $t_d = 6-14$ hr

Table 2a. CONDITIONAL STATISTICS OF RAINSTORM
PARAMETERS FOR DIFFERENT DURATIONS
 $t_d = 1$ hr

Parameter	Mean	Standard Deviation	Min	Max
t_b , hr	55.6	94.5	1	744
t_d , hr	1	0	1	1
D , in.	0.07	0.15	0.01	1.03
D , mm	1.78	3.81	0.25	26.2
i , in./hr	0.07	0.15	0.01	1.03
i , mm/hr	1.78	3.81	0.25	26.2
σ_d , in.	0	0	0	0
σ_d , mm	0	0	0	0
$\bar{t}$, hr	0.50	0	0.50	0.50
G , hr ²	0.33	0	0.33	0.33
a , hr	0.50	0	0.50	0.50
b , hr	0.50	0	0.50	0.50
h , in.	0.14	0.29	0.02	2.06
h , mm	3.56	7.37	0.51	52.3
σ_d^o	0	0	0	0
$\bar{t}^o$	0.50	0	0.50	0.50
G^o	0.33	0	0.33	0.33
a^o	0.50	0	0.50	0.50
b^o	0.50	0	0.50	0.50
h^o	2	0	2	2

Number of Rainstorms = 176

Table 2b. CONDITIONAL STATISTICS OF RAINSTORM
PARAMETERS FOR DIFFERENT DURATIONS
 $t_d = 2$ hr

Parameter	Mean	Standard Deviation	Min	Max
t_b , hr	67.8	76.1	1	327
t_d , hr	2	0	2	2
D , in.	0.24	0.34	0.02	2.09
D , mm	6.10	8.64	0.51	53.1
i , in./hr	0.12	0.17	0.01	1.05
i , mm/hr	3.05	4.32	0.25	26.7
σ_d , in.	0.07	0.12	0	0.87
σ_d , mm	1.78	3.05	0	22.1
$\bar{t}$, hr	0.92	0.27	0.53	1.48
G , hr ²	1.18	0.54	0.40	2.29
a , hr	0.76	0.81	-0.41	2.43
b , hr	1.24	0.81	-0.43	2.41
h , in.	0.24	0.34	0.02	2.09
h , mm	6.10	8.64	0.51	53.1
σ_d^o	0.23	0.15	0	0.48
$\bar{t}^o$	0.46	0.13	0.27	0.74
G^o	0.29	0.13	0.10	0.57
a^o	0.38	0.40	-0.20	1.22
b^o	0.62	0.40	-0.22	1.20
h^o	1	0	1	1

Number of Rainstorms = 134

Table 2c. CONDITIONAL STATISTICS OF RAINSTORM
PARAMETERS FOR DIFFERENT DURATIONS
 $t_d = 3$ hr

Parameter	Mean	Standard Deviation	Min	Max
t_b , hr	54.3	70.2	1	304
t_d , hr	3	0	3	3
D , in.	0.36	0.33	0.03	1.44
D , mm	9.14	8.38	0.76	36.6
i , in./hr	0.12	0.11	0.01	0.48
i , mm/hr	3.05	2.79	0.25	12.2
σ_d , in.	0.09	0.10	0	0.53
σ_d , mm	2.29	2.54	0	13.5
$\bar{t}$, hr	1.38	0.43	0.56	2.40
G , hr ²	2.58	1.28	0.48	5.97
a , hr	1.14	1.29	-1.34	4.18
b , hr	1.86	1.29	-1.18	4.34
h , in.	0.24	0.22	0.02	0.96
h , mm	6.10	5.59	0.51	24.4
σ_d^o	0.23	0.11	0	0.45
$\bar{t}^o$	0.46	0.14	0.19	0.80
G^o	0.29	0.14	0.05	0.66
a^o	0.38	0.43	-0.45	1.40
b^o	0.62	0.43	-0.40	1.45
h^o	0.67	0	0.67	0.67

Number of Rainstorms = 64

Table 2d. CONDITIONAL STATISTICS OF RAINSTORM
PARAMETERS FOR DIFFERENT DURATIONS
 $t_d = 4$ and 5 hr

Parameter	Mean	Standard Deviation	Min	Max
t_b , hr	55.7	64.9	1	292
t_d , hr	4.32	0.47	4	5
D , in.	0.65	0.61	0.08	2.86
D , mm	16.5	15.5	2.03	72.6
i , in./hr	0.15	0.14	0.02	0.72
i , mm/hr	3.81	3.56	0.51	18.3
σ_d , in.	0.15	0.15	0.01	0.53
σ_d , mm	3.81	3.81	0.25	13.5
$\bar{t}$, hr	1.97	0.60	0.66	3.42
G , hr ²	5.17	2.57	0.75	12.4
a , hr	1.58	1.73	-2.02	5.30
b , hr	2.74	1.77	-1.30	6.29
h , in.	0.30	0.28	0.04	1.43
h , mm	7.62	7.11	1.02	36.3
σ_d^o	0.20	0.08	0.06	0.39
$\bar{t}^o$	0.46	0.13	0.17	0.78
G^o	0.28	0.13	0.05	0.65
a^o	0.37	0.40	-0.51	1.33
b^o	0.63	0.40	-0.33	1.51
h^o	0.47	0.05	0.4	0.5

Number of Rainstorms = 50

Table 2e. CONDITIONAL STATISTICS OF RAINSTORM
PARAMETERS FOR DIFFERENT DURATIONS
 $t_d = 6-14$ hr

Parameter	Mean	Standard Deviation	Min	Max
t_b , hr	61.5	77.8	1	324
t_d , hr	8.55	2.62	6	14
D , in.	1.35	0.85	0.30	3.52
D , mm	34.3	21.6	7.62	89.4
i , in./hr	0.16	0.10	0.05	0.53
i , mm/hr	4.06	2.54	1.27	13.5
σ_d , in.	0.15	0.13	0.03	0.74
σ_d , mm	3.81	3.30	0.76	18.8
$\bar{t}$, hr	3.83	1.70	1.00	7.68
G , hr ²	22.0	17.6	2.54	71.2
a , hr	2.93	3.42	-4.99	10.0
b , hr	5.61	3.31	-2.10	13.8
h , in.	0.32	0.21	0.10	1.07
h , mm	8.13	5.33	2.54	27.2
σ_d^o	0.12	0.06	0.04	0.27
$\bar{t}^o$	0.44	0.13	0.13	0.77
G^o	0.27	0.13	0.05	0.68
a^o	0.32	0.39	-0.62	1.30
b^o	0.68	0.39	-0.30	1.62
h^o	0.25	0.07	0.14	0.33

Number of Rainstorms = 31

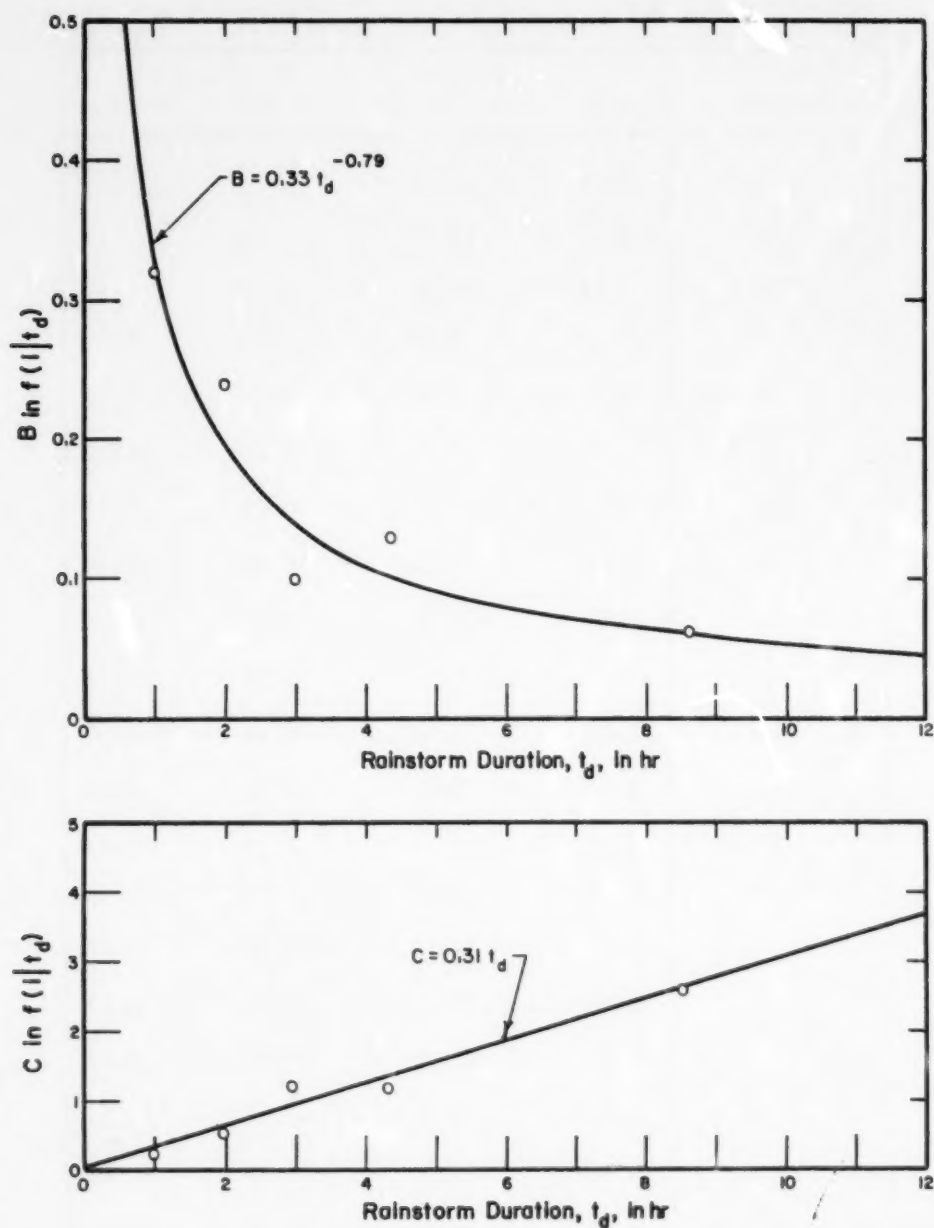


Fig. 8. Gamma density function parameters as functions of rainstorm duration

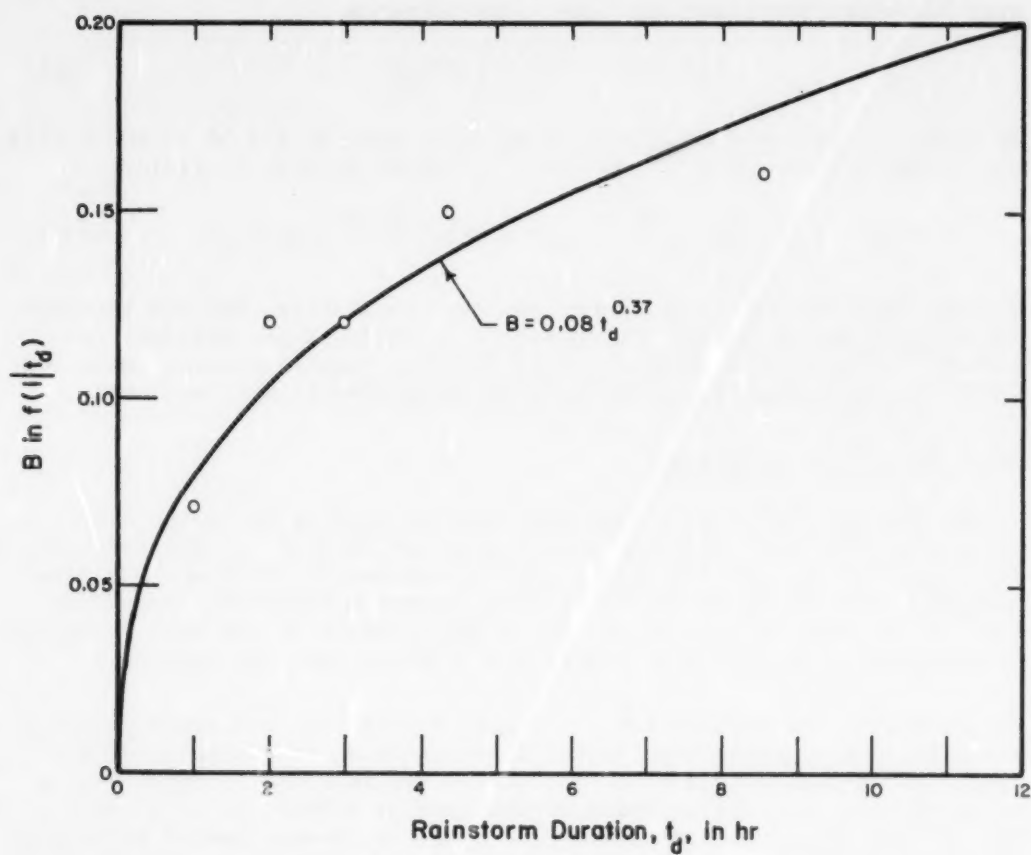


Fig. 9. Exponential density function parameter as function of rainstorm duration

and from Eqs. 20 and 30, the conditional exponential density function is

$$f(i|t_d) = 12.5t_d^{-0.37} \exp(-12.5it_d^{-0.37}) \quad (31)$$

The conditional probability analysis can be extended to obtain joint density functions for practical uses. For instance,

$$f(i, t_d) = f(i|t_d) f(t_d) \quad (32)$$

By using the exponential density functions, Eqs. 31 and 32 together with Eq. 20 (with value of $B = 2.45$ for $f(t_d)$ given in Fig. 4) yield

$$f(i, t_d) = 5.1t_d^{-0.37} \exp(-12.5it_d^{-0.37} - 0.41t_d) \quad (33)$$

Similar analyses can be performed on other parameters, and the procedure can be extended to trivariate cases, e.g., $f(t|D, t_d)$. However, tri-variate frequency analysis is rather tedious, requiring large amount of data, and at present is unlikely to be undertaken by most engineers.

(H) Application Procedure

In urban storm runoff problems, two types of application often arise in connection with the statistical analysis just described. The first is for design of certain facilities such as treatment plants and overflow devices. The second is for operational purposes involving the prediction of the time of occurrence, depth and duration of the next rainstorm after a rainstorm of a given depth and duration has just occurred.

To illustrate the application to design, assume that the storm runoff quantity is the controlling factor in determining the capacity of a wastewater treatment plant and that the plant is to be designed with a capacity which will be exceeded on the average at most twice a month. For the 3-month summer rainstorm data over the 14-year period at Urbana, the average number of rainstorms for a 3-month summer period is $455/14 = 32.5$. Assuming that the Urbana data is applicable for the design under consideration, the given exceedance frequency of twice a month (6 times in 3 months) corresponds to an exceedance probability of $6/32.5 = 0.185$ and a non-exceedance probability of 0.815. The non-exceedance probabilities for rainstorm depth and elapse time between rainstorms as shown in Figs. 10 and 11 can be obtained by integrating the areas of the corresponding curves in Figs. 3 and 6, respectively. It is obvious that among rainstorms of a given frequency those with large depth of rainfall and short duration are most critical to the design. Assuming that depth is the most significant one among all the rainstorm parameters considered, from the non-exceedance probability curve (Fig. 10) for the

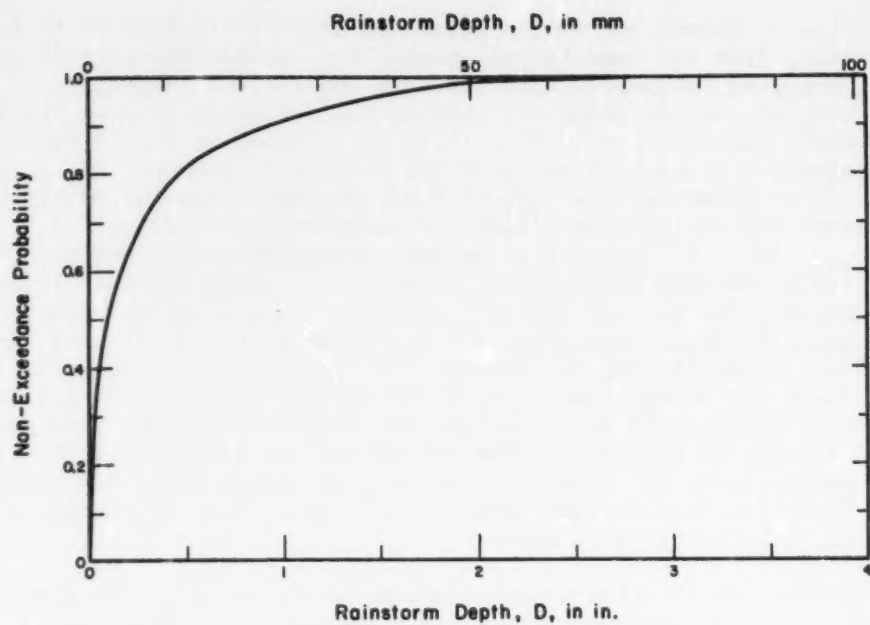


Fig. 10. Non-exceedance probability for rainstorm depth

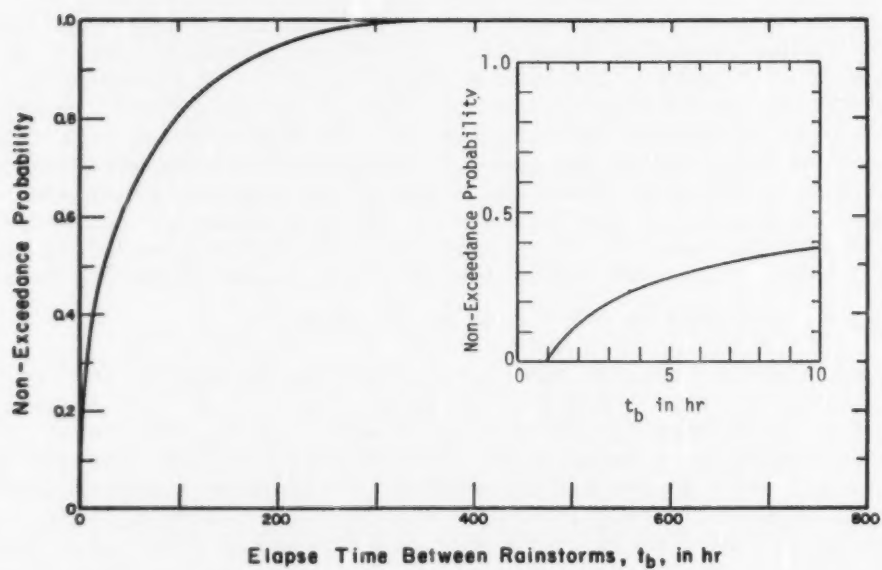


Fig. 11. Non-exceedance probability for elapse time between rainstorms

given design frequency the design rainstorm depth is 12.9 mm (0.51 in.). Subsequently, from the conditional probability density function of t_d , $f(t_d|D)$, the most frequently occurred duration for this depth, i.e., the mode of $f(t_d|D)$, can be found and used as the design duration. For the Urbana summer data for $D = 12.9$ mm (0.51 in.), based on the 49 rainstorms with D between 8.9 and 16.5 mm (0.35 and 0.65 in.), the mode of t_d is 2 hr (Fig. 12). Likewise, the mode of $\bar{t}$ can be found from the conditional probability density function $f(\bar{t}|D)$, or alternatively, $f(\bar{t}|t_d)$ or $f(\bar{t}|D, t_d)$. Because $\bar{t}$ appears to be more sensitive to t_d than to D , and because the trivariate conditional probability density function is difficult to obtain, the mode of $f(\bar{t}|t_d)$ is adopted as the design value of $\bar{t}$. For the example Urbana data (Fig. 13) the value of $\bar{t}$ used for the design is 1.025 hr. Substituting the values of D , t_d , and $\bar{t}$ into Eqs. 14, 15, and 16 yields the design values of the triangular hyetograph shape factors a , b , and h , being 1.075 hr, 0.925 hr, and 12.9 mm (0.51 in.), respectively. The design hyetograph thus determined can subsequently be routed through the surfaces and sewers of the drainage basin using the routing methods that will be described later to give the design discharge or hydrograph for the treatment plant.

For the case of application to storm water runoff control, the problem is of the nature of flow prediction for management purposes. For instance, for an existing drainage system, when a rainstorm comes, it is desirable for operational purposes to know the time of occurrence, depth and duration of the next rainstorm so that decision can be made on the utilization of in-line storage and other control facilities. Consider the case when a rainstorm of depth D and duration t_d has just occurred. Data from Urbana and other locations indicate that the elapse time between rainstorms is nearly independent of the depth and duration of the preceding rainstorm. Therefore, assuming that t_b is independent of D and t_d , the most probable time of occurrence of the next rainstorm can be estimated as the mode of the probability density function such as the one shown in Fig. 6. The depth of the next rainstorm can be evaluated as the mode of the conditional distribution of the depth of a rainstorm given the depth of the previous rainstorm, $f(D_2|D_1)$. Subsequently the duration of the next rainstorm can be determined from the mode of the conditional probability density function $f(t_d|D_2)$ and the shape of the hyetograph for the next rainstorm from $f(\bar{t}|t_{d2})$ as described in the design application.

Of course, many refinements and improvements can be made on the procedures for runoff control and for design just described. For example, in the flow prediction for storm water runoff control, the elapse time t_b can be estimated by using a non-exceedance probability function for t_b (Fig. 11) with an assumed or selected non-exceedance probability. The depth of the next rainstorm can be estimated by using $f(D_2|D_1, t_{d1})$ instead of $f(D_2|D_1)$, and $\bar{t}$ by $f(\bar{t}|t_{d2}, D_2)$ instead of $f(\bar{t}|t_{d2})$. The corresponding risks of the prediction can be evaluated accordingly using the joint density functions of the parameters involved. However, such refined methods are rather tedious and complicated, and available data are often inadequate to establish the needed conditional probabilities.

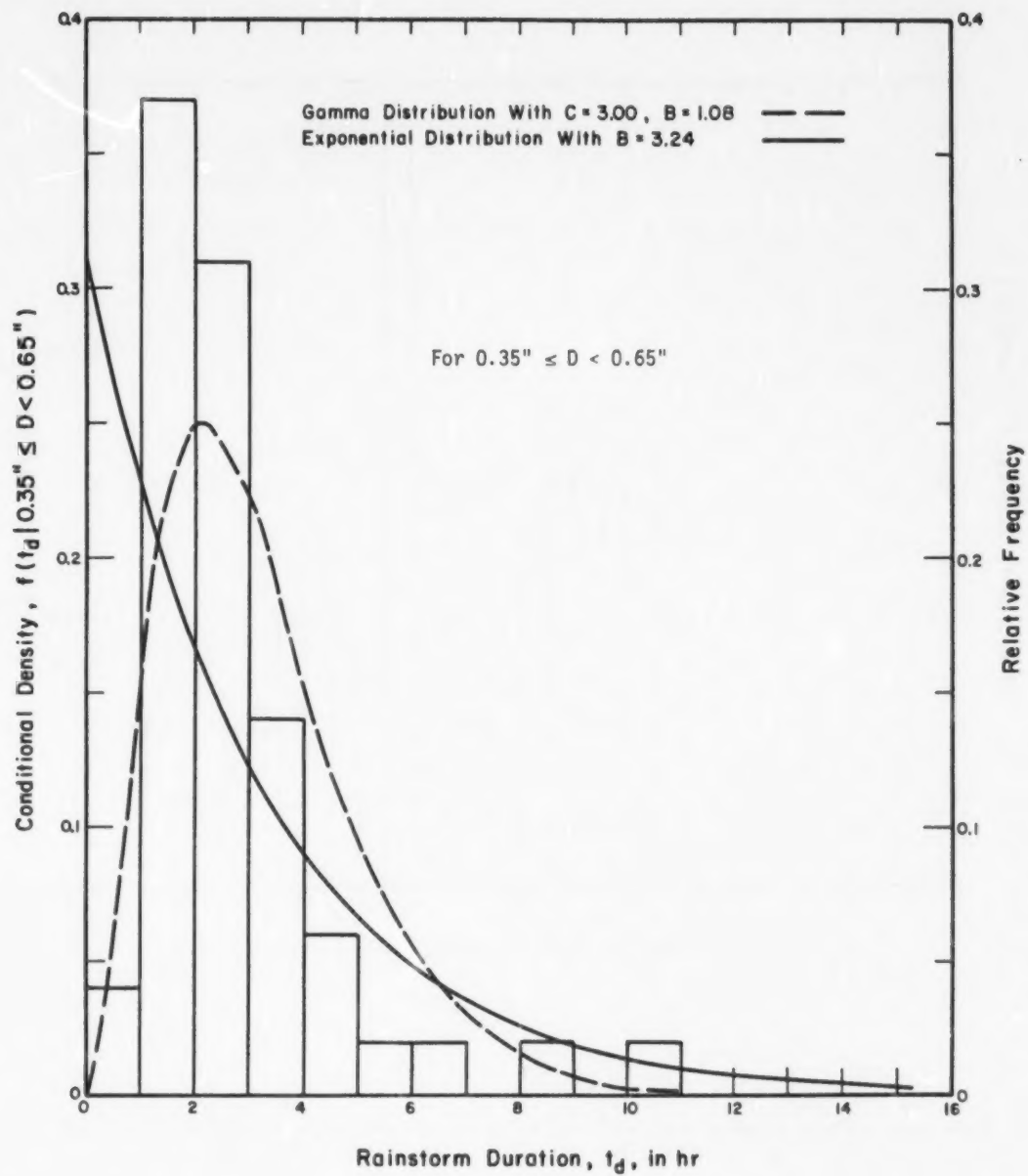


Fig. 12. Conditional distribution of rainstorm duration

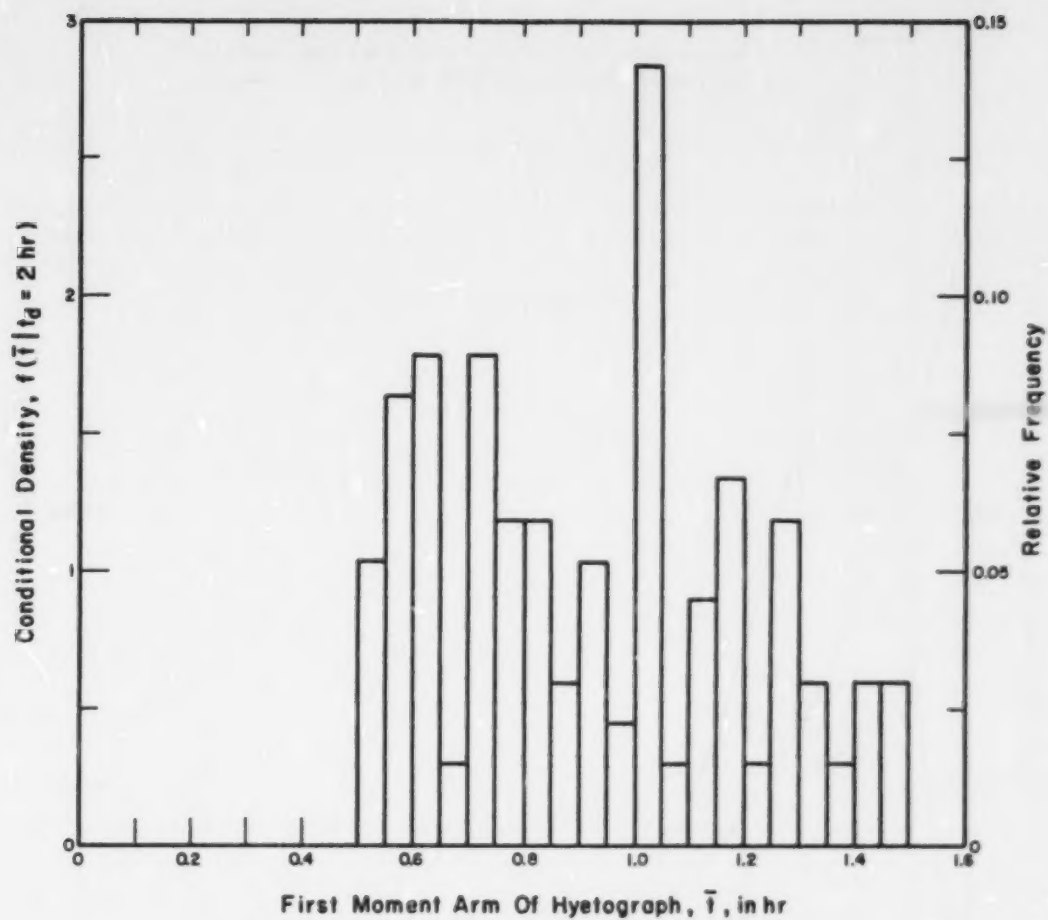


Fig. 13. Conditional distribution of first moment arm of hyetograph for two-hour duration and for all rainstorm depth

Therefore, they are suggested to be considered only in the future after significant information based on simpler procedures are obtained and when adequate data are available. Moreover, if weather forecasting is available, it will provide more reliable information on t_b and D_2 than that given by Fig. 6 and $f(D_2|D_1)$. Consequently the conditional probability analysis should be carried out taking into account the information from weather forecasting.

IV-2. INFILTRATION AND OTHER ABSTRACTIONS

Not all of the rainfall produces runoffs. In hydrology the losses that do not produce surface runoff are called abstractions. These losses consist of interception, evaporation, transpiration, infiltration, and depression storage. Interception is the amount of rainwater being intercepted by trees, vegetation, posts and buildings that never reaches the ground surface. For urban areas the rainfall on roofs which is drained to the surface or directly to the sewers is not considered an interception. The relative importance of interception on runoff depends on the intensity and duration of rainfall. In urban areas usually there is no dense woods or vegetation, the amount of interception is no more than a fraction of an inch (a few mm) and mostly occurs during the beginning of the rainfall. Therefore, for relative heavy rainfall of short duration, which would be of importance for design or for pollution control because of overflow, the amount of interception is less than a few percent of the runoff volume and can be neglected without causing serious accuracy problems.

Evaporation and transpiration are often considered simultaneously for obvious practical reasons. Evapotranspiration may be important when the water balance over a long period is considered. But as shown by Shen et al. (1974), it is negligible when heavy rainfall over short duration is considered, particularly in view of the vegetation and tree situation in most urban areas. Actually, there is no difficulty to include interception and evapotranspiration losses when reliable formulas become available. At present the amount of these losses is assumed negligible.

Infiltration loss is a major factor affecting surface runoff. Infiltration is defined as the process of water flowing through the ground surface, i.e., the interface between the fluid environment and the soil environment below. Various theoretical approaches and empirical formulas have been proposed to estimate infiltration. At the beginning of this research project a study was made to use Philip's (1969) theory to derive a four-parameter method to account for infiltration. Unfortunately, available field data are inadequate to substantiate this approach and hence the simpler and popular Horton's formula is used. In fact, even for Horton's formula which is a three-parameter function, there are difficulties to establish the values of the parameters based on existing data. Philip (1969) also proposed a two-parameter approximate infiltration equation. However, this equation has not been adequately tested nor has it been widely accepted by civil engineers.

Horton's formula is

$$f = f_c + (f_o - f_c) e^{-kt} \quad (34)$$

in which f is the instantaneous infiltration capacity; f_o and f_c are the initial and final infiltration capacities, respectively; t is time; and k is an exponent accounting for the decay rate of infiltration. For Horton's equation to apply, the water supply rate (rainfall and water stored on land surface) must be equal to or greater than the infiltration capacity at that instant. Otherwise, the entire amount of water is assumed infiltrated.

The difficulty in applying Horton's formula to actual drainage basins arises partly from the fact that in experimental and field rainfall-runoff studies, the soil properties and surface moisture conditions are often inadequately recorded, and partly due to the fact that for a natural drainage basin the soil condition is inevitably nonhomogeneous and the values of f_c , k and f_o vary within the drainage basin. The measured basin runoff hydrograph merely reflects the integrated effects of infiltration and other factors, and there is actually no single set of representative values of f_o , f_c , and k for the entire basin.

Theoretically, strictly speaking, for a given soil none of the values of k , f_c , and f_o is constant. They depend on the soil type, fluid properties, moisture condition of the soil, and water pressure (usually depth) on the ground surface. The initial infiltration capacity, f_o , obviously depends heavily on the initial soil moisture condition. The final infiltration rate, f_c , and the exponent expressing the decay rate, k , are the soil properties and should be constant if secondary effects such as those due to changes in soil flow potential near the ground surface, in water depth, in fluid properties and seasonal effects are neglected. It has shown in Figs. 16 and 17 of Philip (1969) that for a given soil and liquid, f_c is essentially constant but the decay rate decreases with decreasing initial moisture and with increasing overlying water depth, i.e., k decreases with decreasing initial moisture content or increasing water depth. However, for a given surface in the practical range of conditions the variation of k is relatively small. In other words, from a purely theoretical viewpoint considering a liquid entering a porous medium, the values of f_c and k are not constant, but from a practical viewpoint in using Horton's formula, the values of f_c and k can be treated as essentially constants. The approximate constancy of f_c for a given soil surface has generally been accepted. The relative constancy of k is still a matter of debate. Many researchers using experimental data tried to show that k varies considerably with initial soil moisture condition and other factors. However, a small error in measurement in f_c would easily give a varying k for the same soil. Of course, the differences may also reflect the seasonal effects. Unfortunately, for the purpose of the present study on storm runoff this

uncertainty on infiltration imposes a serious problem on the accuracy of the results, making a reliable comparison of the runoff prediction methods difficult. Should Horton's formula be used with f_c and k treated as constants, it is suggested that different sets of values be used for different seasons.

A simple one-parameter approach, the ϕ -index method has also been used for rainfall-runoff studies. The method assumes a constant infiltration rate over the period of rainfall. This method is incompatible with the requirement for the more sophisticated urban runoff methods evaluated in this study.

It should be noted here that not all the infiltrated water is necessarily lost because some water may find its way through subsurface flow to contribute to the basin runoff. In urban basins this may occur as infiltrated water entering sewers through joints and other leakages. However, such a case is not considered in this study.

The amount of loss due to depression storage depends on how the term is defined. Loosely it is usually defined as the water to fill the ground depressions before surface runoff starts. The amount, obviously, is a function of the surface texture. Actually, as it is commonly defined, the depression storage includes a thin layer of water held by surface tension before surface runoff starts. Expectedly the depression storage is of statistical nature. The most commonly used formula for depression storage supply rate (in./hr or mm/hr) is (Linsley et al., 1949)

$$s = (i - f) \exp \left[\frac{-(P-F)}{s_c} \right] \quad (35)$$

in which s_c is the depression storage capacity expressed in depth (in. or mm); P is the cumulative rainfall in depth; and F is the cumulative infiltration in depth.

IV-3. SNOW MELT

Runoff from snow melt in urban areas differs from that of rural areas in two major aspects. First there is more heat available for snow melting in urban areas than in rural. Second and most important, the intense human activities and interferences in urban areas would hasten the melting process. Although runoff from snow melt is never a problem in urban storm sewer design because its magnitude is smaller than the flash flood of urban runoff due to heavy rainstorms, it is of considerable importance in pollution control because of the quality of the melted water, particularly when de-icing additives are used to speed up the melting process.

The energy needed for snow melting comes mainly from three sources: the radiant heat from the sun, the conduction heat from the environment, and the latent heat of vaporization released by the condensation of water

vapor. The first two are the major ones to be considered for urban snow melting. Many empirical and semi-empirical studies have been made on snow melt in rural areas (Chow, 1964, Section 10). However, these studies consider a time of melting usually much longer than that which would be interesting and useful for urban settings, and the interferences of human activities are not included. Snow melt in urban areas is a topic practically untouched. A possible approach is to consider the energy budget of snow melting and the thermodynamic processes involved, such as the idea outlined by Eagleson (1970, Chap. 13). But such an idea has not been extended or developed into any form nearly adoptable in practice, and to undertake such a research is beyond the scope of this study.

From an engineering viewpoint the worst condition in terms of both runoff quantity and quality for snow melt is melting of snow under warm rain. For this situation the following daily snow melt formula recommended by the U.S. Army Corps of Engineers (1960) is tentatively adopted in this study:

$$M = 0.007 P_r (T_a - 32) \quad (36)$$

in which M is the daily snow melt in in., P_r is the daily rainfall in in., and T_a is the mean daily temperature of saturated air at 10-ft above ground in °F. If M and P_r are in mm and T_a in °C, the formula becomes

$$M = 0.013 P_r T_a \quad (37)$$

SECTION V

DRAINAGE SYSTEM CHARACTERISTICS OF OAKDALE AVENUE BASIN

The Oakdale Avenue Drainage Basin in Chicago was selected to verify and evaluate the urban storm runoff models. It is one of the very few drainage basins for which relatively compatible and reliable data are available. The basin is located in a residential section in the city of Chicago (Fig. 14). It is approximately 2-1/2 block long by 1 block wide and has a drainage area of 0.052 km² or 12.9 acres. A typical photographic view of the basin is shown in Fig. 15. The basin consists entirely of residential dwellings, and the drainage characteristics are relatively uniform. The street pattern may be considered as being typical of many cities in Illinois and the United States.

V-1. SURFACE DRAINAGE PATTERN

The drainage pattern of the land surface of the Oakdale Basin is shown in Fig. 16. There are 30 inlet catch basins, each delivering its water to a sewer junction and each receiving water from a gutter except inlets 7, 14, 15, 21, 29, and 30 (Fig. 16) which receive water from two gutters for each of these 6 inlets. Each of the 36 gutters is contributed by water from one or more subcatchments as shown by the dashed lines in Fig. 16. The subcatchment area of the Oakdale Basin consists of four types of surfaces: roofs, lawns, paved sidewalks, and street pavements. The relative percentages of size of these four types of surfaces vary from subcatchment to subcatchment, and as one would expect, also change with time as a result of change of land uses and structures. Detailed data on the distribution of these four types of surfaces for each of the subcatchments in the Oakdale Basin is not available.

In most metropolitan areas, data on such detailed land surface uses are generally nonexistent. For a small drainage basin like the Oakdale Basin, it is possible to conduct a detailed survey to actually measure the physical properties of each of the subcatchments. However, even for such a small basin, at least several man-months of work is needed to obtain the data. Atop of this difficulty there is always the problem of getting permission to survey in private properties. One simpler, faster, and less costly method is to use aero-photos or satellite pictures. Because of the time limitation of the research project, this photogrammetric method was not undertaken. Nevertheless, even if such detailed surface use data is available, it would be extremely tedious and costly to actually use it to route the rainwater through all the surfaces to the gutters. In view of the practical consideration of the costs involved in obtaining and using the detailed subcatchment surface data and the seasonal changes of the surface characteristics, it appears most unlikely that any practical urban storm runoff simulation model would require using such detailed subcatchment information. For the case of the

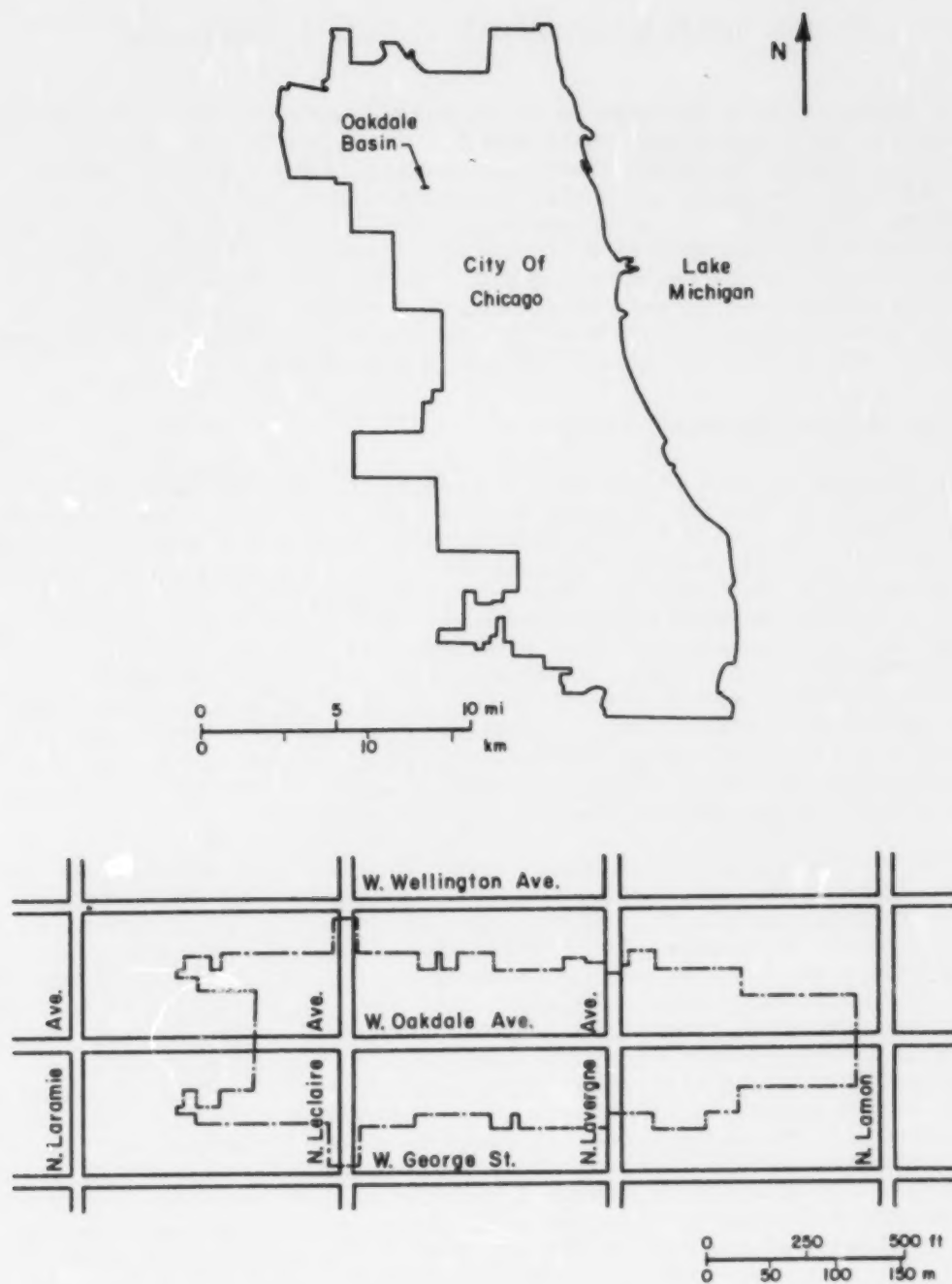


Fig. 14. Oakdale Basin location map



(a) Oakdale Avenue



(b) Alley

Fig. 15. Typical photographic view of Oakdale Basin
(Courtesy of Illinois State Water Survey)

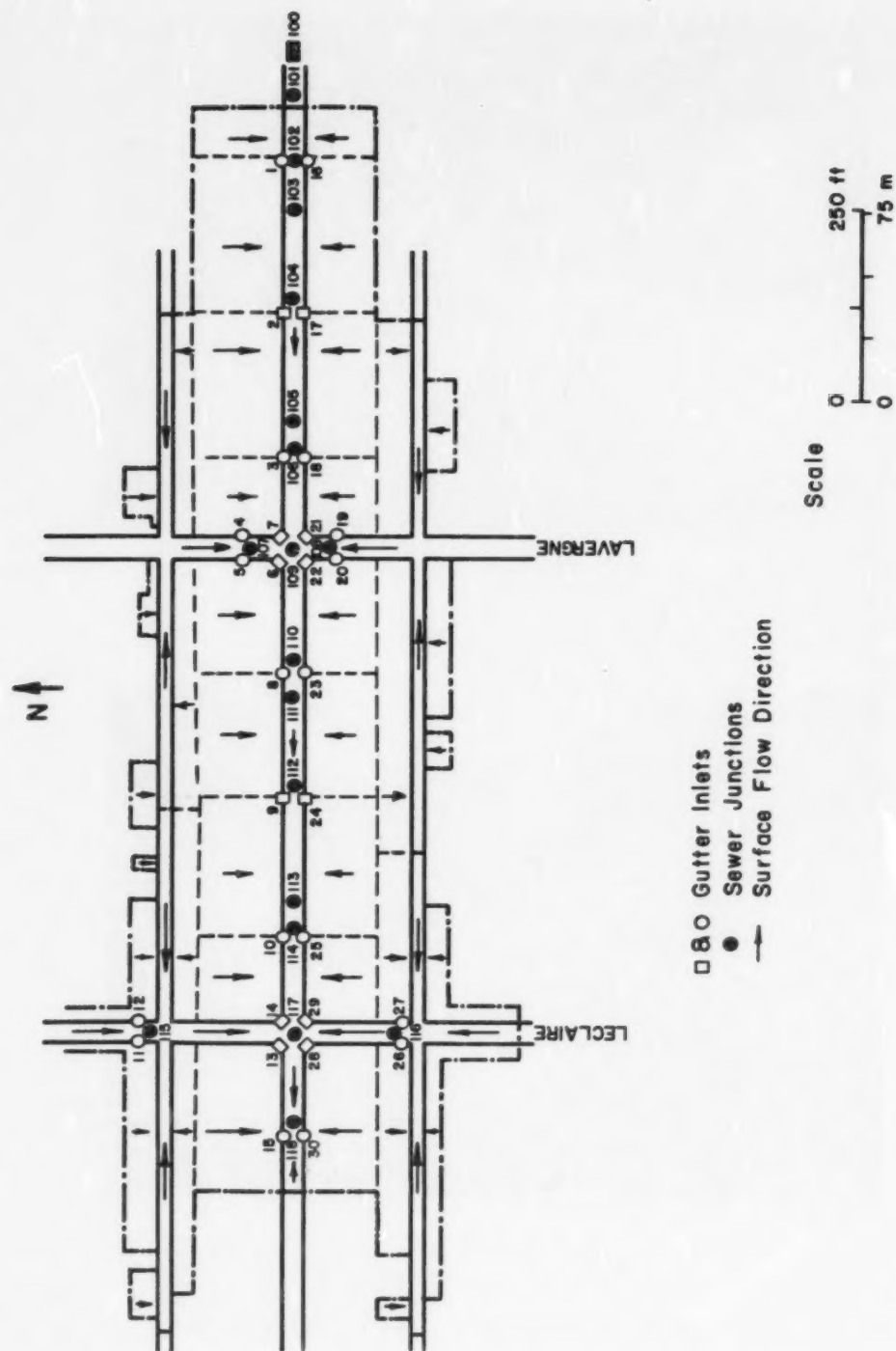


Fig. 16. Drainage pattern of Oakdale Basin

Table 3. DIMENSIONS OF GUTTERS OF OAKDALE AVENUE DRAINAGE BASIN

Gutter From Inlet	To Inlet	Gutter Length		Size of Contributing Subcatchments		Longitudinal Slope	Type of Grate Inlet at Downstream End of Gutter*	Contribution to Sewer Junction
		ft	m	ac	m ²			
-	1	58	18	0.17	690	0.0012	C	102
1	2	201	61	0.53	2140	0.0012	R	104
2	3	192	59	0.51	2060	0.0012	C	106
3	7	120	37	0.32	1290	0.0012	R	109
-	4	104	32	0.04	160	0.0010	C	107
-	5	104	32	(0.54) #	(2190) #	0.0010	C	107
4	7	46	14	0.03	120	0.0010	R	109
5	6	42	13	0.02	80	0.0010	R	109
6	8	117	36	0.32	1290	0.0027	R	110
8	9	194	59	0.51	2060	0.0027	R	112
9	10	200	61	0.53	2140	0.0027	C	114
10	14	114	35	0.31	1250	0.0027	R	117
-	11	100	30	0.02	80	0.0010	C	115
-	12	100	30	0.02	80	0.0010	C	115
11	13	192	59	0.07	280	0.0010	R	117
12	14	192	59	(0.50) #	(2020) #	0.0010	R	117
13	15	96	29	0.26	1050	0.0010	C	118
-	15	96	29	0.26	1050	0.0010	C	118
-	16	58	18	0.17	690	0.0012	C	102
16	17	201	61	0.53	2140	0.0012	R	104
17	18	192	59	0.51	2060	0.0012	C	106
18	21	120	37	0.32	1290	0.0012	C	109
-	19	100	30	0.01	400	0.0010	C	108
-	20	100	30	(0.54) #	(2190) #	0.0010	C	108
19	21	42	13	0.02	80	0.0010	R	109
20	22	42	13	0.02	80	0.0010	C	109
22	23	117	36	0.32	1290	0.0027	C	110
23	24	194	59	0.51	2060	0.0027	R	112
24	25	200	61	0.53	2140	0.0027	C	114
25	29	114	35	0.31	1250	0.0027	R	117
-	26	151	46	0.13	530	0.0010	C	116
-	27	151	46	(0.64) #	(2590) #	0.0010	C	116
26	28	128	39	0.05	200	0.0010	R	117
27	29	128	39	0.05	200	0.0010	R	117
28	30	96	29	0.26	1050	0.0010	C	118
-	30	96	29	(0.41) #	(1660) #	0.0010	C	118

*Type of grate inlets: C = circular, R = rectangular

#Contribution by alleys

Gutter width W = 1.0 ft (0.30m)

Gutter bottom inclined 88° 15' from vertical curb

Manning's n for gutters = 0.013

Oakdale Basin used in this research, although the instrumented survey was not extended to cover the details of the subcatchments, a thorough visual survey was conducted in order to provide reliable information for the evaluation of the urban storm runoff methods.

The streets in the Oakdale Basin are 8.5 m (28 ft) wide, paved with asphalt having a cross-slope of 2.4 to 3.0% on both sides of the street crown. The longitudinal slope of the streets varies as listed in Table 3. The street slopes are typically flat as for most of the Midwest cities. Between Leclaire Street and the outlet of the basin Oakdale Avenue is actually sloping down towards west although the sewer line underneath has its slope in the opposite direction.

The street gutters are all triangular in cross section formed by cast-in-place concrete. The curbs are essentially vertical and the curb height varies along the gutters and often interrupted by driveways. The most frequently observed curb height is 0.25 m (10 in.). The lateral angle, θ , between the gutter bottom and the vertical is 1.54 radians. There is a break of lateral slope where the gutter bottom joins the street pavement. Consequently, assuming that the water surface of the gutter flow is horizontal along the lateral direction, within the gutter width W (Fig. 17) the relationship between the flow cross sectional area, A , and the depth measured from the apex of the gutter, h , is

$$A = \frac{h^2}{2 \cot \theta} \quad \text{for } h \leq W \cot \theta \quad (38a)$$

$$A = Wh - \frac{W^2 \cot \theta}{2} \quad \text{for } h > W \cot \theta \quad (38b)$$

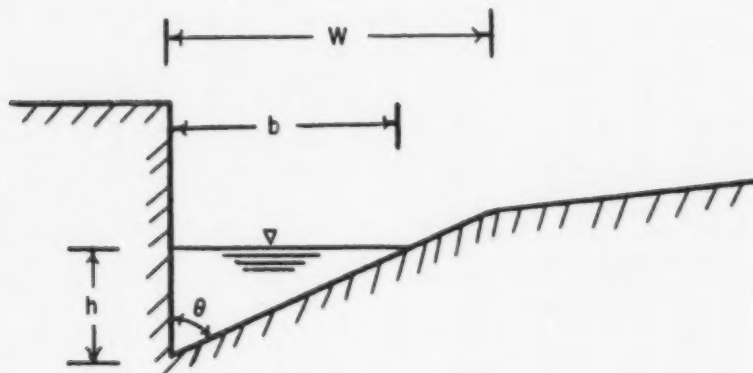


Fig. 17. Schematic drawing of gutter cross section

The hydraulic radius R is

$$R = \frac{h \sin \theta}{2(1 + \cos \theta)} \quad \text{for } h \leq W \cot \theta \quad (39a)$$

$$R = \frac{Wh - \frac{W^2 \cot \theta}{2}}{h + \frac{W}{\sin \theta}} \quad \text{for } h > W \cot \theta \quad (39b)$$

and the water surface width b is

$$b = \frac{h}{\cot \theta} \quad \text{for } h \leq W \cot \theta \quad (40a)$$

$$b = W \quad \text{for } h > W \cot \theta \quad (40b)$$

The longitudinal slopes of the gutters are the same as those for the streets. The gutter width measured horizontally is 0.3 m (1 ft). The geometric dimensions of the 36 gutters are listed in Table 3. In most of the time the gutters are kept reasonably clean although some debris have been observed. Accordingly Manning's roughness factor n is estimated to be 0.013 for the gutters.

A certain amount of rainstorm water is discharged directly from subcatchments into the alleys between the streets as shown in Fig. 16. Hydraulically these alleys act like wide shallow channels to transport the water into inlets or gutters. Most of the alleys have concrete surface with uneven joints and cracks and their estimated Manning's roughness factor n is 0.016. The length, width, and slope of the alleys are listed in Table 4.

The 30 inlets in the basin are grate inlets either circular or rectangular in shape as listed in Table 3. The details for the circular grate inlets are shown in Fig. 18a and those for the rectangular in Fig. 18b. The approximate locations of the inlets are identified by the inlet numbers in Fig. 16. The distances between the inlets are given in Table 3. Some of the inlets do not start from the curb line but offset slightly and extend beyond the gutter proper into the street pavement. Such irregularity occurs mostly for replacing inlets with clogged inlet catch basins. Apparently some of the inlet catch basins have the clogging problem. There is no record to identify whether the inlet catch basins surveyed in 1973-74 were the same as those a decade ago.

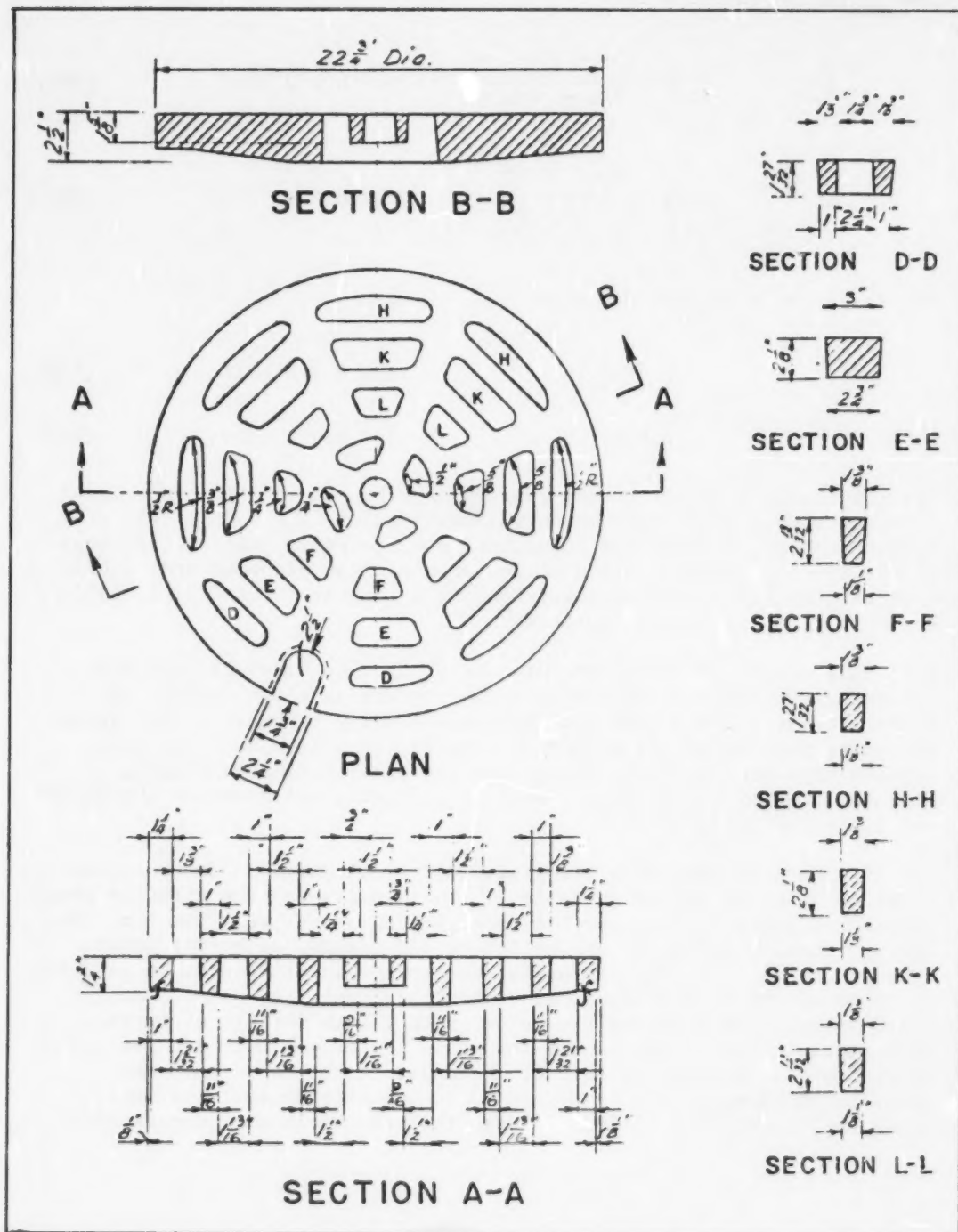


Fig. 18a. Details of grate inlets, Circular grate inlet

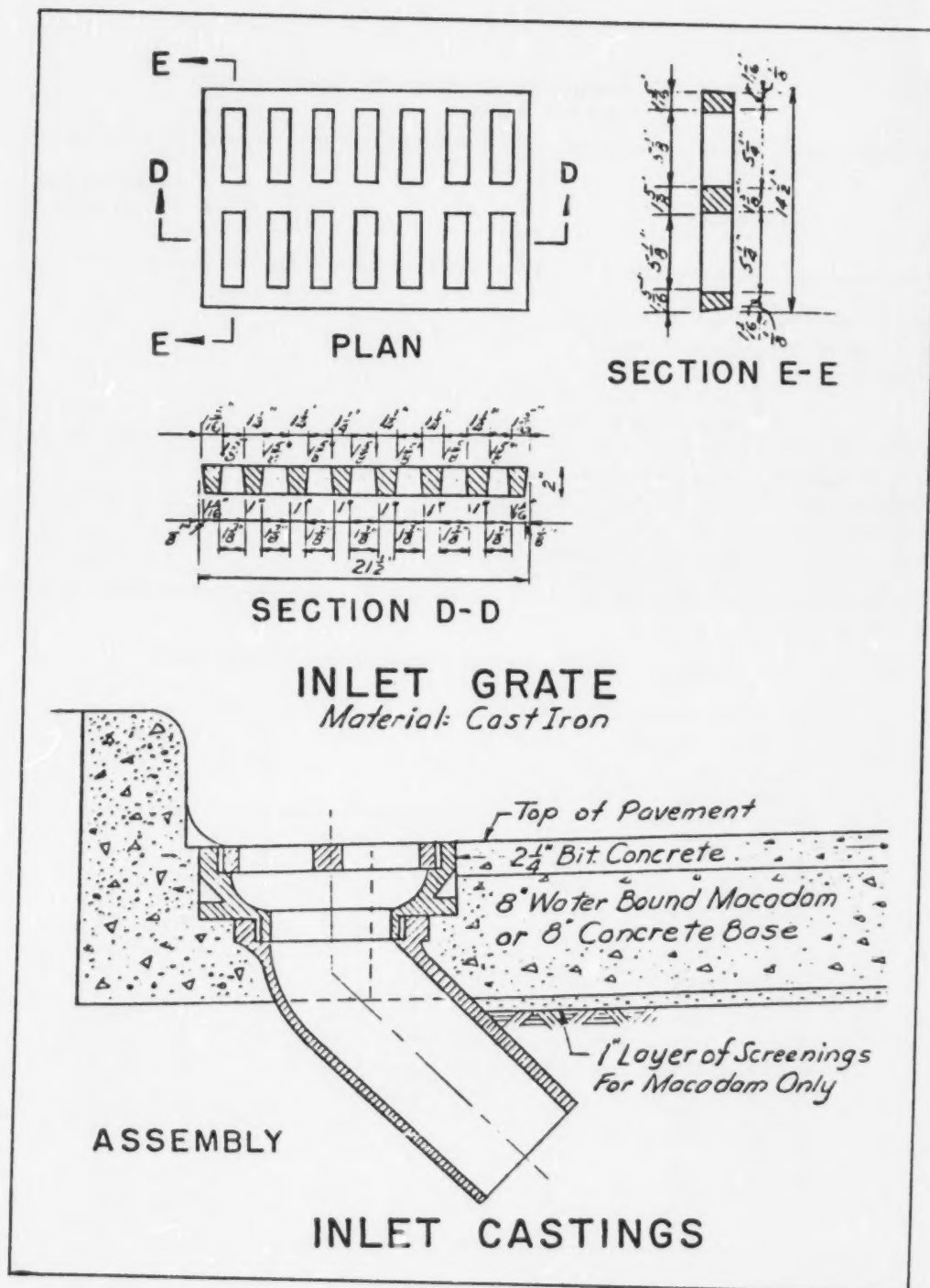


Fig. 18b. Details of grate inlets, Rectangular grate inlet

Table 4. DIMENSIONS OF ALLEYS OF OAKDALE AVENUE
DRAINAGE BASIN

Location		Length		Width		Slope	Contributing to inlet
		ft	m	ft	m		
Alleys between Wellington and Oakdale	West of Leclaire	395	120	15.5	4.7	0.0047	13
	East of Leclaire	295	90	15.5	4.7	0.0049	14
	West of Leclaire	335	121	15.5	4.7	0.0042	5
	East of Lavergne	295	90	15.5	4.7	0.0053	4
Alleys between Oakdale and George	West of Leclaire	396	121	15.5	4.7	0.0043	26
	East of Leclaire	236	72	15.5	4.7	0.0053	27
	West of Lavergne	380	116	15.5	4.7	0.0040	20
	East of Lavergne	290	88	15.5	4.7	0.0054	19

Table 5. DIMENSIONS OF SEWERS OF OAKDALE AVENUE
DRAINAGE BASIN

Sewer From To Node Node		Length		Slope %	Diameter	
		ft	m		ft	m
118	117	108	32.9	0.72	1.00	0.30
115	117	170	51.8	0.71	0.83	0.25
116	117	105	32.0	1.08	0.83	0.25
117	114	134	40.8	0.45	1.25	0.38
114	113	34	10.4	0.45	1.25	0.38
113	112	168	51.2	0.45	1.25	0.38
112	111	158	48.2	0.40	1.50	0.46
111	110	38	11.6	0.40	1.50	0.46
110	109	131	39.9	0.40	1.50	0.46
107	109	50	15.2	3.78	0.83	0.25
108	109	45	13.7	4.20	0.83	0.25
109	106	153	46.6	0.35	1.75	0.53
106	105	39	11.9	0.35	1.75	0.53
105	104	156	47.6	0.35	1.75	0.53
104	103	156	47.6	0.30	2.00	0.61
103	102	61	18.6	0.30	2.00	0.61
102	101	73	22.3	0.30	2.00	0.61
101	100	32	9.8	0.30	2.50	0.76

V-2. SEWER SYSTEM

The combined sewer system of the Oakdale Avenue Drainage Basin consists of 18 circular sewer pipes and 18 junctions or manholes plus the sewer system outlet. The diameter of the concrete pipes ranges from 0.25 m (10 in.) to 0.76 m (30 in.) and the pipe roughness is estimated to be 0.01 ft or 3 mm. The junctions or manholes are marked as 3-digit numbers (e.g., 101 to 118) in Fig. 16 and the dimensions of the sewers are listed in Table 5. Most of the manholes are of flow-through type with a half-cut pipe embedded at the manhole bottom connecting the upstream and downstream sewers to induce smooth flow at low discharge. None of the junctions or manholes has a horizontal cross sectional area bigger than 2 m² (20 sq ft) and hence their storage capacity is relatively small.

For circular sewers such as those used in the Oakdale Basin, when the pipe is flowing partially filled with a depth h and central angle $\theta = 2 \cos^{-1}[1 - (2h/D)]$ as shown in Fig. 19, the flow area A and the corresponding hydraulic radius R and water surface width b are

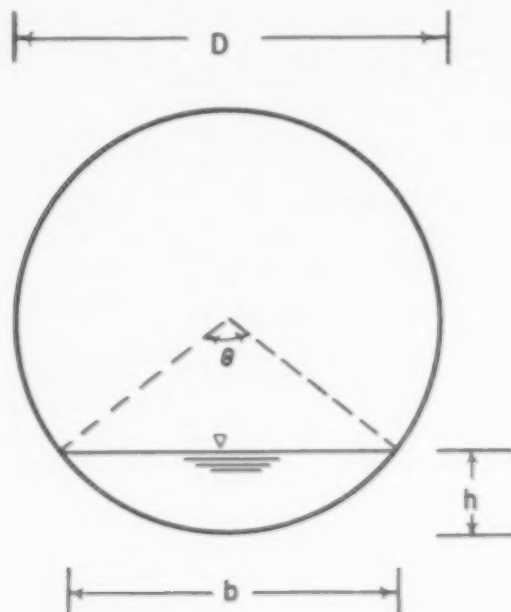


Fig. 19. Circular sewer flow cross section

$$A = \frac{D^2}{8} (\theta - \sin\theta) \quad (41a)$$

$$R = \frac{D}{4} \left(1 - \frac{\sin\theta}{\theta}\right) \quad (41b)$$

$$b = D \sin\frac{\theta}{2} \quad (41c)$$

for $0 \leq \theta \leq 2\pi$ and D is the pipe diameter.

At the outlet of the Oakdale Avenue Drainage Basin a Simplex 0.76 m (30 in.) Type "S" parabolic flume is placed in a vault at the corner of Oakdale and Lamon Avenues to measure and record the basin runoff. This runoff measurement and recording system together with a tipping bucket recording rain gauge located at one block north of the basin has been in operation since 1959 measuring rainfalls and runoffs. Details of these measuring devices and the data collected can be found elsewhere (Tucker, 1968) and are not presented here.

SECTION VI

ILLINOIS SURFACE RUNOFF MODEL

The Illinois Urban Storm Runoff (IUSR) method actually consists of three parts: the surface runoff model, the sewer system hydraulic routing model and the sewer system water quality model. The input into the surface runoff model is the hyetographs. The output is the inlet hydrographs, which constitute the input into the sewer system routing model, as well as the inlet pollutographs if required and provide the pollutant input is given. The sewer routing model is the Illinois Storm Sewer System Simulation Model (Sevuk et. al., 1973) and will be described briefly in Section VII. The sewer system water quality model which will be described in Section VIII takes the output pollutographs from the surface runoff model and the computed hydrographs from the sewer routing model as its input to produce pollutographs at any desired locations of the sewer system. A block diagram showing schematically the relationship between these three models is given in Fig. 20. However, any one of these three component models can be used independently to interface with other runoff models provided the data requirement is compatible. The Illinois Surface Runoff model is a recent development and will be discussed in this section. It should be noted here that improvement and refinements are continuously being made on all the three models and those reported here are the most up-to-date versions at the time of writing this report.

VI-1. RUNOFF IN SUBCATCHMENTS

The land surface of a drainage basin producing runoff can be subdivided into two subsequent parts: the subcatchments which consists of strips of overland flows receiving rainfall as the input; and the gutters which receive water from the subcatchments as well as from direct rainfall and deliver the water into inlet catch basins to produce inlet hydrographs. The overland surface of a drainage basin can be approximated by a number of constant-width rectangular strips of different lengths. A large number of such strips of narrow width will closely approximate the actual overland surface. But this will require a large amount of computations without significant improvement in accuracy. Contrarily, too few strips to grossly simplify the representation of the complicated conditions of actual overland surfaces consisting of roofs, lawns, sidewalks, drive-ways, etc., approximate the actual geometry poorly.

Time varying free-surface flow including overland flows can be described mathematically by a pair of partial differential equations called the St. Venant equations (Chow, 1959; Yen, 1973b)

$$\frac{\partial h}{\partial t} + D \frac{\partial V}{\partial x} + V \frac{\partial h}{\partial x} = \frac{1}{b} \int_{\sigma} q d\sigma \quad (42)$$

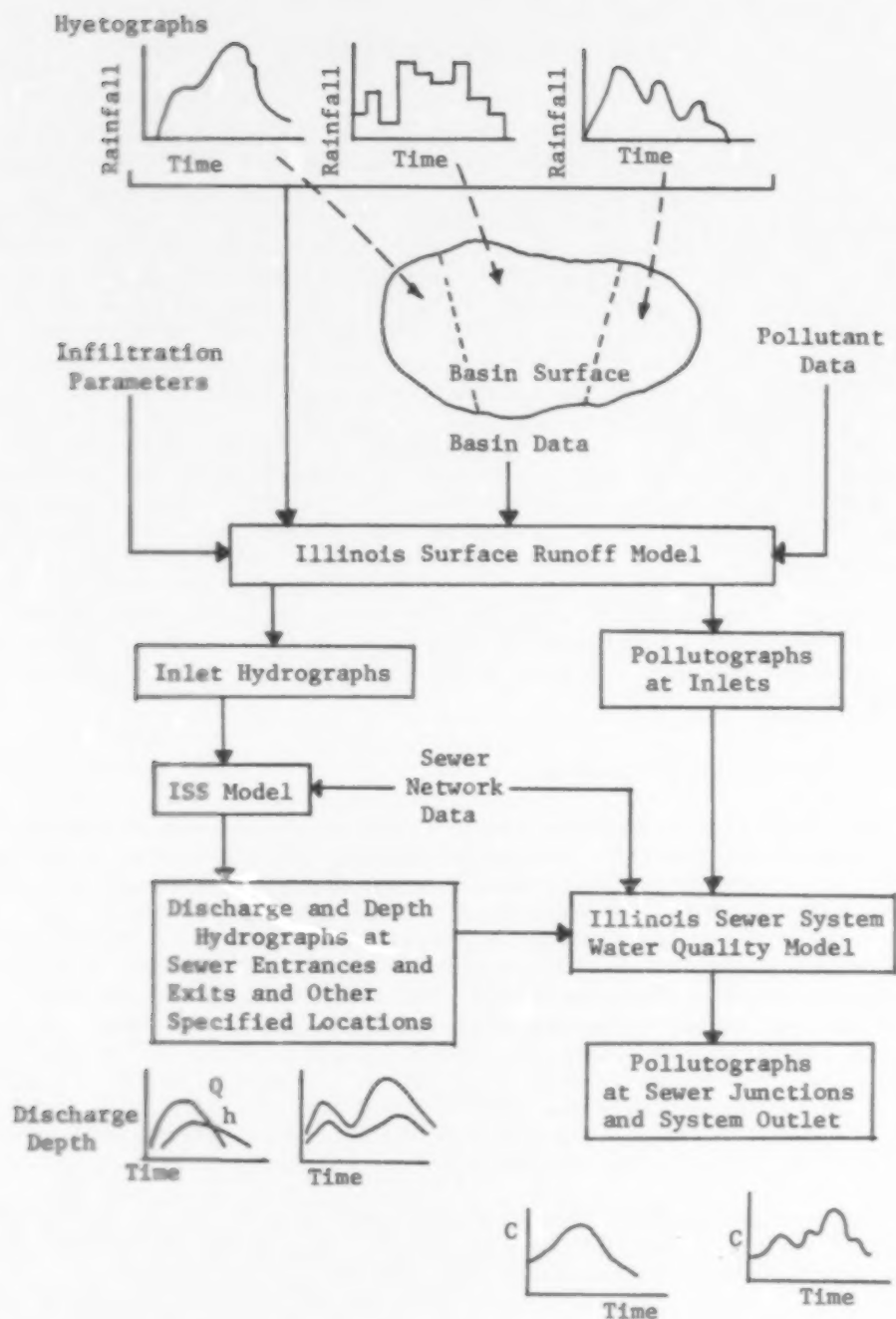


Fig. 20. Block diagram of Illinois Urban Storm Runoff Model

$$\frac{\partial V}{\partial t} + V \frac{\partial V}{\partial x} + g \cos \theta \frac{\partial h}{\partial x} = g(S_o - S_f) + \frac{1}{A} \int_{\sigma} (U_1 - V) q \, d\sigma \quad (43)$$

in which x is the direction of the flow measured along the bed; t is time; A is the flow cross sectional area; b is the width of the free surface; $D = A/b$ is the hydraulic depth; V is the cross sectional average flow velocity; h is the depth of the flow above the invert; θ is the angle between the channel bed and the horizontal; $S_o = \sin \theta$ is the bed slope; S_f is the friction slope; σ is the perimeter bounding A ; q is the lateral discharge per unit length of σ having a velocity component U_1 along the x -direction when joining or leaving the flow, being positive for inflow and negative for outflow; and g is the gravitational acceleration (Fig. 21). The first equation is the equation of continuity and the second the momentum equation.

With the appropriate initial and boundary conditions, these two equations can be solved numerically using digital computer for the overland flow on subcatchments. However, the solution requires considerable amount of computer time and in view of the usually large number of subcatchments (overland strips) such an approach is feasible but impractical. In field conditions, the accuracy of data on overland geometry and rainfall input usually does not render the accuracy that the St. Venant equations can provide. Several approximations of Eqs. 42 and 43 are possible. Often the overland flow is approximated by using Manning's formula or Izzard's method which essentially assumes the flow or rainfall to be steady. Other approximations of the St. Venant equations include the kinematic wave model and diffusion wave model (Yen, 1973a). From past experience the nonlinear kinematic wave model is found to be most suitable for solving overland flows because it does not require a downstream boundary condition and hence considerably reduces the computer time and difficulties and yet its accuracy is substantially better than that given by Manning's formula. Those who are interested in the relative accuracy of the different approximate models can refer elsewhere (Sevuk, 1973; Yen, 1973a).

For the kinematic wave approximation, the inertia and pressure terms of the momentum equation (Eq. 43) are neglected; thus,

$$S_o = S_f \quad (44)$$

The friction slope S_f can be estimated by using the Darcy-Weisbach formula

$$S_f = f \frac{1}{4R} \frac{V^2}{2g} \quad (45)$$

in which f is the Weisbach resistance coefficient given by the Moody diagram and R is the hydraulic radius. Alternatively, S_f can be estimated by the Manning formula

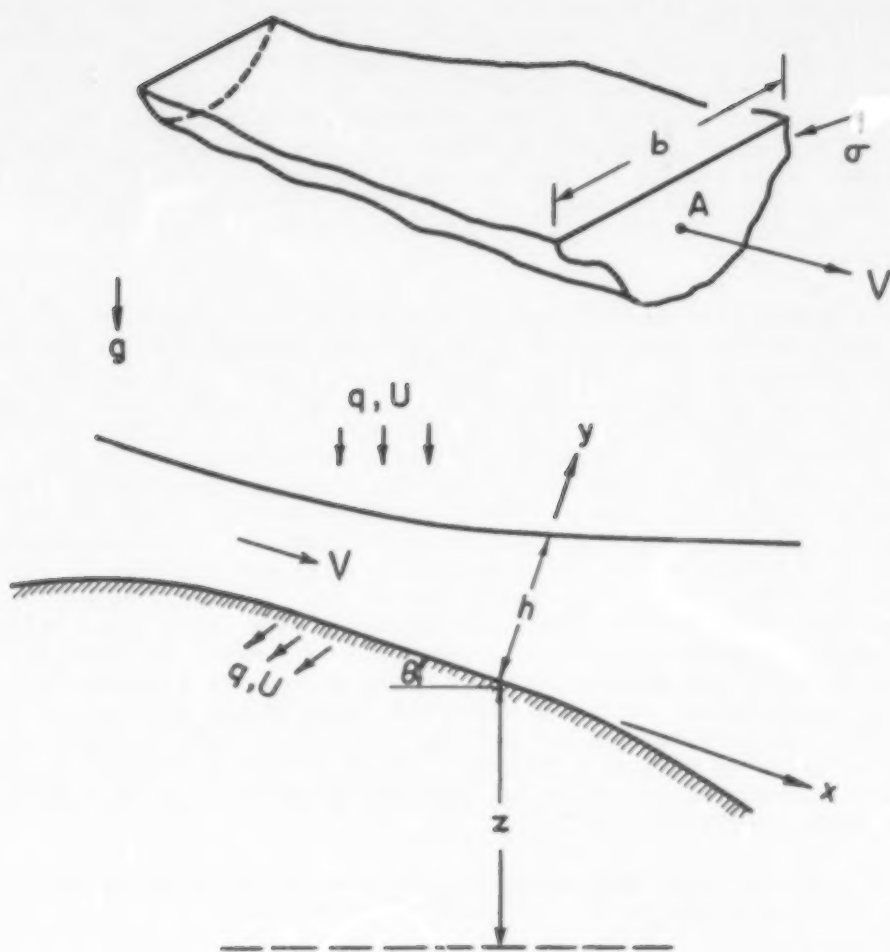


Fig. 21. Definition sketch of open-channel flow

$$S_f = \frac{n^2}{2.22} V^2 R^{-4/3} \quad (46)$$

where n is Manning's roughness factor; or by the Chezy formula

$$S_f = V^2 / c^2 R \quad (47)$$

in which c is the Chezy factor. The continuity equation (Eq. 42) and Eq. 44, together with the initial condition and one upstream boundary condition, can be solved numerically for the unsteady flow. Equation 46 is for V in fps and R in ft; if V is in m/sec and R in m, the coefficient is unity instead of 2.22.

In selecting the resistance formula to approximate the friction slope, the Weisbach coefficient has the advantage of being dimensionless and having better theoretical justification, whereas Manning's n has the advantage of being nearly constant independent of flow depth for flows over rough boundaries with sufficiently high Reynolds number. However, for overland flows the depth is usually so shallow and the Reynolds number of the flow not sufficiently high that it would be erroneous to consider n to be constant (Yen, 1975). Therefore, the Weisbach formula (Eq. 45) is adopted in this model to evaluate the overland flow.

In Eq. 45, the value of f is given by the Moody diagram which can be found in standard hydraulics reference books (e.g., Rouse, 1950; Chow, 1959). For the case of overland flow under rainfall, limited information was given by Yen et al. (1972) and Shen and Li (1973). Based on the available information, the Weisbach f is computed as

$$f = C/R \quad (48)$$

for laminar flow, in which $R = VR/\nu$ is the Reynolds number of the flow where ν is the kinematic viscosity; and the coefficient C is

$$C = 24 + 101 i^{0.4} \quad (49a)$$

for i in mm/hr, or

$$C = 24 + 27 i^{0.4} \quad (49b)$$

for i in in./hr. Since the surface of natural overland is inevitably rough, for turbulent flow, f is computed as

$$\frac{1}{\sqrt{f}} = 2 \log \frac{2R}{k} + 1.74 \quad (50)$$

where k is a length measure of surface roughness. The transition between Eqs. 48 and 50 is shown schematically in Fig. 22. The critical Reynolds number R_c determining which equation should be used is

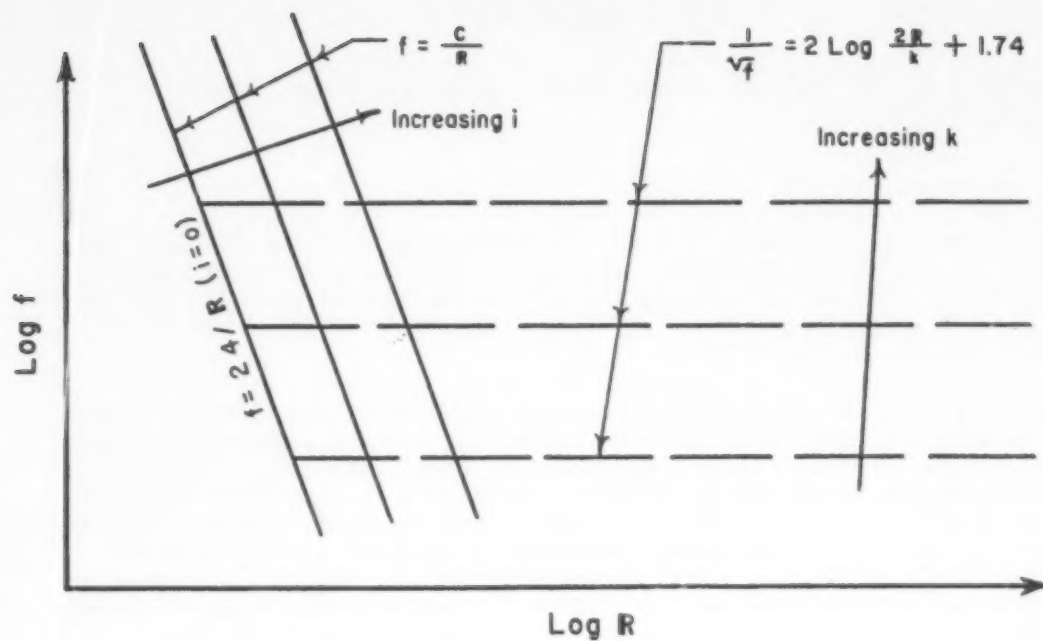


Fig. 22. Evaluation of Weisbach resistance coefficient

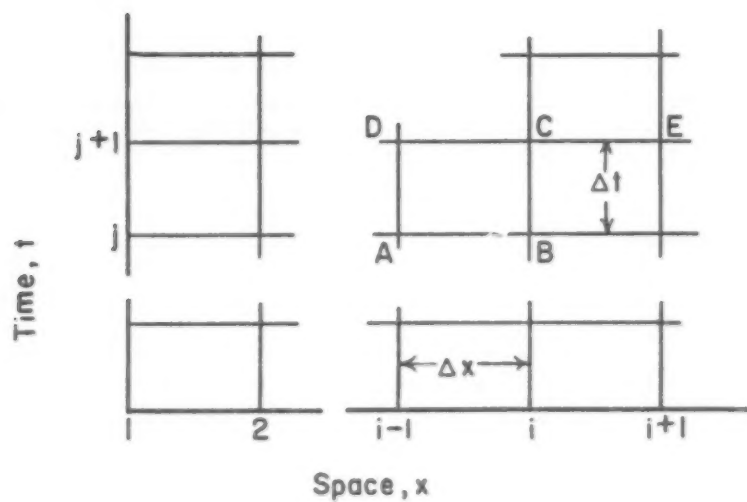


Fig. 23. Computational grid for semi-implicit four-point backward difference scheme

$$R_c = C \left(2 \log \frac{2R}{k} + 1.74 \right)^2 \quad (51)$$

When the Reynolds number of the flow $R < R_c$, Eq. 48 applies. Otherwise, Eq. 50 is used. It should be noted that actually there is a transition between laminar (Eq. 48) and fully developed rough turbulent flow (Eq. 50). This transition is neglected here and the steady uniform flow values of f are used for unsteady cases. This approximation can of course be improved when more information on f becomes available.

The water input onto the subcatchment surface to produce the flow is the lateral flow q in Eqs. 42 and 43. The value of q is equal to the rainfall minus infiltration. The infiltration is estimated by using Horton's formula (Eq. 34). When the rainfall rate is smaller than the infiltration capacity, the deficiency is supplemented by the water on the surface, if any.

In solving Eqs. 42, 44 and 45 numerically, the initial condition to start the solution cannot be zero depth and zero velocity because this condition will impose a mathematical singularity. In reality, when rain falls on a dry overland surface, there is indeed an initial wetting process before runoff starts. The surface tension will hold a small amount of water without producing runoff. Therefore, the initial condition for the overland runoff from the subcatchments can be assumed as a small finite depth with zero velocity. In other words, immediately following the commencement of rainfall, after infiltration and other losses are subtracted, the water left on the overland surface simply accumulated without producing runoff until the initial depth is reached. This initial depth depends on the slope and nature of the overland surface. Future studies will provide more information on this initial depth. It suffices at present to assume the initial depth to be 0.0012 in. or 0.03 mm. It has been found that the final solution is practically unaffected by the value of the initial depth so long as it is assumed within a reasonable realistic range.

Several numerical schemes can be used to obtain the solution (Sevuk and Yen, 1973; Price, 1974). A 4-point, noncentral, semi-implicit scheme is used to solve the equations because of its independent selection of the time and space increments (Δt and Δx) in the computations without stability problems and consequently saves computer time.

A more accurate and convenient form of Eq. 42 for the purpose of numerical solution is

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q_l \quad (52)$$

where Q is the flow rate at any flow section, A is the flow cross-sectional area and $q_l = \int_{\sigma} q d\sigma$ is the lateral flow per unit length of flow in x -direction, being positive for inflow. Applying the chain rule

of differentiation to Eq. 52, and letting $G(h) = \partial Q / \partial h$ and $b(h) = \partial A / \partial h$, one obtains

$$G \frac{\partial h}{\partial x} + b \frac{\partial h}{\partial t} = q_L \quad (53)$$

For the semi-implicit four-point backward difference scheme adopted, referring to the fixed rectangular grid in Fig. 23, the coefficients and partial differential terms of Eq. 53 may be approximated by the following expressions

$$G = \frac{1}{2} (G_D + G_C) \quad (54a)$$

$$b = \frac{1}{2} (b_D + b_C) \quad (54b)$$

$$\frac{\partial h}{\partial x} = \frac{1}{\Delta x} (h_C - h_D) \quad (55a)$$

$$\frac{\partial h}{\partial t} = \frac{1}{2\Delta t} (h_D + h_C - h_A - h_B) \quad (55b)$$

Substitution of Eqs. 54 and 55 into Eq. 53 yields

$$\frac{1}{2\Delta x} (G_C + G_D) (h_C - h_D) + \frac{1}{4\Delta t} (b_D + b_C) (h_D + h_C - h_A - h_B) = q_L \quad (56)$$

The flow parameters at grid points A and B are known either from the initial conditions or from previous time step computations, and the flow parameters at point D are known either from the upstream boundary condition or from previous computations. Therefore, with G_C and b_C being specified functions of h_C , the only unknown in Eq. 56 is h_C .

Surface runoff on subcatchments usually occurs in the form of open-channel flow in wide channels. Consequently the runoff problem can be simplified by solving for the discharge per unit overland width, Q_u . Hence, in Eq. 56, $b_A = b_B = b_C = b_D = 1$. For laminar flow in a wide rectangular channel, combining Eqs. 44, 45 and noting that $Q_u = VR$ and $R = h$, one obtains

$$Q_u = \frac{8gS_o}{Cv} h^3 \quad (57)$$

Hence

$$G = \frac{\partial Q_u}{\partial h} = \frac{24gS_o}{Cv} h^2 \quad (58)$$

For the case of turbulent flow, Eqs. 45 and 50 yield

$$Q_u = \sqrt{8gS_o} \left(2 \log \frac{2h}{k} + 1.74 \right) h^{3/2} \quad (59)$$

and

$$G = \sqrt{8gS_o} \left(3 \log \frac{2h}{k} + 3.47 \right) h^{1/2} \quad (60)$$

The depth of water h at any subcatchment flow section is obtained by using Eqs. 56 and 58 for laminar flow, and Eqs. 56 and 60 for turbulent flow. Newton's iteration technique is used for the numerical processes. Knowing the flow depth, the discharge per unit width is evaluated by using Eq. 57 and Eq. 59 respectively, for laminar and turbulent flows. After the flow parameters at grid point C is computed, the computations for the next downstream station at the same time level (grid point E in Fig. 23) can be performed. After the flow parameters at all the stations at a given time level are evaluated, the computations is advanced to the next time level starting from the upstream end.

VI-2. GUTTER FLOW ROUTING

Street gutters and surface runoff in defined channels receive water from overland runoff of subcatchments, from upstream water sources, if any, and directly from rainfall. The input water is transported through the gutter or channel into the inlet catch basin to produce the inlet hydrographs for the sewer runoffs. Horton's formula (Eq. 34) is assumed applicable to account for infiltration. Theoretically, the gutter flow is also described mathematically by the St. Venant equations (Eqs. 42 and 43). Again, using these equations to solve for gutter flows is feasible but impractical in view of the large number of gutters for a drainage basin and the accuracy of the input data. In field conditions gutters are rarely prismatic channels because of poor control in construction and interruption of local facilities such as driveways and other intersections. Furthermore, in actual operation, gutters are often obstructed by debris, parked cars and the like. Such obstructions are time varying and random in nature. It is impossible to describe all the boundary conditions of the gutters precisely for any particular runoff considered. Therefore, solving the gutter runoff by using the St. Venant equations, which require large amount of computer time and detailed geometry data, cannot be justified. Consequently the kinematic wave approximation is adopted for gutter flow routing as for the case of overland flow in subcatchments.

The differential equations of the kinematic-wave model for gutter flows are the same as those for overland flows, i.e., Eqs. 42 and 44. Therefore, the same solution technique can be used for both cases, and Eq. 56 is the finite difference equation representing also the gutter flow. However, the flow conditions in gutters are often within the range where Manning's formula (Eq. 46) can be used to approximate the friction slope,

S_f . Use of Manning's formula instead of Darcy-Weisbach's (Eq. 45) simplifies the computation as it is no longer needed to check the instantaneous flow Reynolds number in order to estimate S_f . Thus, from Eqs. 44 and 46,

$$Q = \frac{C_n}{n} S_o^{1/2} AR^{2/3} \quad (61)$$

and

$$G = \frac{C_n}{n} S_o^{1/2} \left(\frac{2}{3} AR^{-1/3} \frac{\partial R}{\partial h} + R^{2/3} \frac{\partial A}{\partial h} \right) \quad (62)$$

where $C_n = 1$ in SI system and 1.49 in English system. The terms A , R , $\partial A / \partial h$ and $\partial R / \partial h$ in Eq. 62 should be evaluated from the cross-sectional shape of the gutter under consideration. For instance, for the triangular gutters in the Oakdale Avenue Drainage Basin (Fig. 16) these terms can be evaluated by using Eqs. 38 and 39. Specifying b in Eq. 56 as a function of h from the gutter geometry, it is possible to solve Eqs. 56 and 62 simultaneously for the flow depth h_c at grid point C using Newton's iteration technique. The discharge then is evaluated from Eq. 61. The computation progresses in the downstream direction for each time level as described for subcatchment runoffs.

The initial condition for the gutter flow routing is essentially the same as that for runoff in subcatchments. As to the boundary conditions, kinematic wave model requires only the upstream conditions be provided. In the present gutter routing model the upstream boundary condition is provided as specified flow depths at the upstream end of the gutter at each time level. These flow depths are evaluated within the model using Manning's formula (Eq. 46) and they correspond to the carry-over of water from the upstream inlets. When there is no such carry-over at the upstream end of a gutter, the depth of water at the upstream flow section is assumed to be always equal to the initial depth.

VI-3. INLETS

Inlets are one of the most important components of urban drainage systems to determine the time distribution of urban storm runoffs. They control the amount of water to flow from gutters into sewers. The hydraulic characteristics of an inlet depend on the geometric properties of the inlet. Unfortunately, despite the large number of inlets used in streets and highways, the geometries of inlets have never been standardized. Furthermore, in their operation, inlets are seldom kept clean to be free from foreign materials partially clogging the inlet.

Inlets can be classified as curb type and grate type. A combination of the two is also used. Hydraulically they can be described by the weir formula

$$Q = C_d b H^{3/2} \quad (63)$$

or the orifice formula

$$Q = C_d A H^{1/2} \quad (64)$$

in which C_d is a discharge coefficient; A is the cross-sectional area of the orifice opening; b is the length of the weir; and H is the available head. The value of H depends on the gutter or surface flow depth near the inlet. The difficulty in using Eqs. 63 and 64 for inlet flow computation is the wide range of variations of the values of C_d and A , particularly for unclean inlets. Also, the determination of the range of application of the weir and orifice formulas is a matter of debate.

The inlet imposes a backwater effect on the gutter flow. For a supercritical flow in the gutter, disturbance waves cannot propagate upstream and hence the numerical solution of the St. Venant equations or its nonlinear kinematic-wave approximation can proceed forward from upstream without depending on the downstream boundary conditions.

For a subcritical flow the gutter flow is directly affected by the hydraulic conditions at its downstream end, i.e., the inlet. Consequently the inlet flow condition, which is by itself unknown and yet to be solved, becomes the necessary downstream boundary condition for the numerical solution of the St. Venant equations. Contrarily, for the kinematic-wave approximation, no downstream boundary condition is required. Consequently, the gutter flow can be solved without requiring simultaneous solution of the yet unknown inlet flow conditions, and hence the solution technique can be simplified and the required computer time greatly reduced. The inlet flow can subsequently be computed as will be described later. Such an approximation neglecting the backwater effect due to the inlet of course differs from the reality. However, in view of the uncertainties on the physical conditions of the gutters and inlets, it appears to be justified from a practical viewpoint that the gutter flow is computed by using the nonlinear kinematic-wave approximation and the inlet flow is computed independently.

A more accurate and potentially practical approach to gutter-inlet flow solution is to use generalized nondimensional curves describing inlet runoff hydrographs for different input and geometry conditions (Akan, 1973). However, this approach requires at least a certain degree of standardization of the inlets in order to avoid a large number of nondimensional graphs and hence it is not adopted here, although it may be used in the future for the refinement of the surface runoff model.

In the Illinois Surface Runoff model, the average depth plus the velocity head at the end of the gutter is used as the value of H in Eqs. 63 and 64 for the calculation of the inlet discharge. The discharge coefficient C_d in these equations is assigned different values according to the type of inlet under consideration. For instance, in Eq. 63, C_d is assumed to be equal to 3.0 for grate inlets with longitudinal bars and for combined inlets, 2.4 for grate inlets with diagonal bars, 2.7

for grate inlets with cross bars, and 1.2 for curb openings. The corresponding C_d values in Eq. 64 are 0.60, 0.48, 0.54, and 0.30, respectively. The inlet discharge is first computed by using Eq. 63 until this equation gives a discharge greater than the discharge of the approaching gutter flow. From then on, it is assumed that the flow around the inlet has the characteristics of orifice flow, and the inlet discharge is computed by using Eq. 64. During the recession of the gutter runoff, when the computed inlet discharge using the weir formula is smaller than the approaching gutter flow, the inlet discharge is assumed to be computed again by using the weir formula, Eq. 63.

When an inlet receives water from more than one gutter, the inlet is assumed subdivided into a number of component inlets each associated with a contributing gutter. Each component inlet is assumed to act independently from others when applying Eqs. 63 or 64.

When the approaching gutter flow is greater than the inlet discharge, the excessive water is assumed carried over the inlet to continue on as the input flow into the next gutter immediately following. Should there exist more than one downstream gutter such as at an intersection, the model allows a distribution of the carry-over flow among these downstream gutters. However, the distribution factors should be provided on the program input data cards. This carry-over of excessive flow from inlet is assumed to continue until the flow reaches a low point such as Junctions 109 and 117 in Fig. 16 where no further carry-over can reasonably be assumed and a reservoir storage routing is performed for the discharge through the last inlet and the storage around it.

The assumptions on the distribution of carry-over flow, on the transition between the weir and orifice flows, and on the values of C_d are not precise as the reality. Improvement and refinements on these aspects can be made in the future when more reliable and useful laboratory and field data become available.

VI-4. COMPUTER PROGRAM DESCRIPTION

The Illinois Surface Runoff Model is programmed in Fortran IV language for computer solutions. The input into the computer program consists of the geometric characteristics of the subcatchments, gutters, inlets and the identification of the sewers joining the inlet catch basins, and also the rainfall hyetographs. The output is the inlet hydrographs which serve as the input into the sewer system. The program also performs water quality computations of the runoff to produce inlet pollutographs. The formulation and details of the water quality part of the model will be given in Section VIII.

The computer program of the Illinois Surface Runoff Model as listed in App. B allows the consideration of a maximum number of 100 gutters at a time. Along each gutter, the subcatchments can be approximated by as many as 10 rectangular strips. These strips of overland areas

may have different lengths, slopes, and surface and infiltration properties, but should be equal in width. Two different pollutants are considered at a time for each gutter and subcatchment strip. Different types of gutters and inlets are considered as identified by the type numbers shown in Fig. 24. The program allows for the entire basin a maximum of five zones of rainfall each with a different hyetograph. The computational logic is shown schematically in Fig. 25. The computer storage requirement for the program in its present form is about 400K. If more storage is available, the program can easily be modified to consider larger basins. This modification can be achieved by simply changing the size of the arrays in the dimension statements.

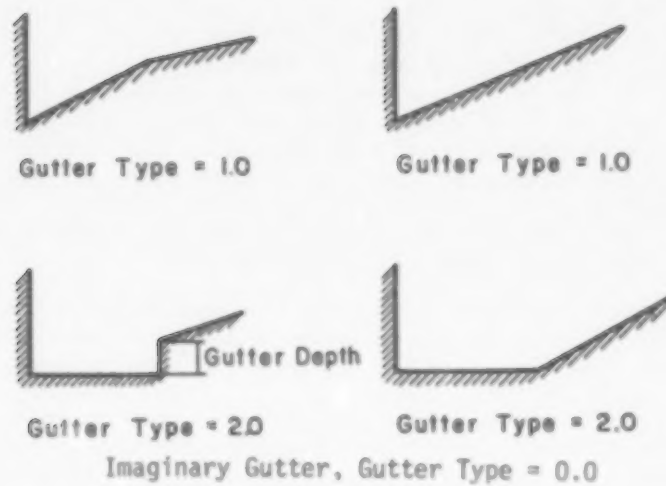
The computer program consists of one main program and six subroutines. The relationship between the main program and the subroutines is shown schematically in Fig. 26. A brief description is as follows:

MAIN PROGRAM: It reads and stores data for the entire basin. It performs the gutter routing computations based on the nonlinear kinematic wave and Manning's equations. Newton's iteration technique is used to solve Eqs. 56 and 62, and discharge is computed by using Eq. 61. The computations are made starting from the most upstream gutters and proceeding towards downstream. A mechanism is built in the program to decide which gutters should be considered first. This allows the gutters in the drainage basin to be numbered arbitrarily from 1 to 100 while preparing data. However, the user should specify the flow direction in each gutter. The water quality computations for gutter flows are also performed in the main program.

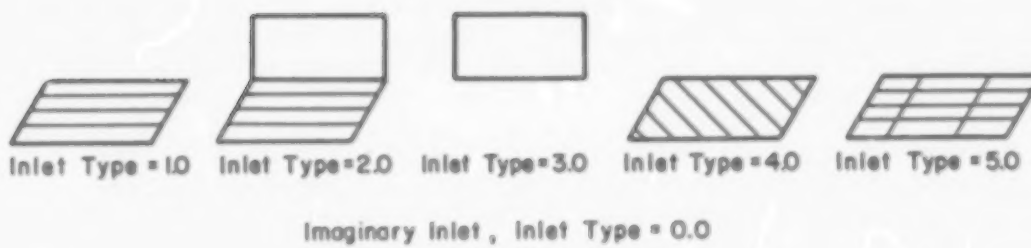
SUBROUTINE OVLFLO: Flow in subcatchment strips is computed in this subroutine. Newton's iteration technique is used to solve the nonlinear kinematic wave equations with the Darcy-Weisbach formula; i.e. Eqs. 56 to 61. An approximate form of the Moody diagram (Fig. 22) is built in the subroutine to estimate the resistance coefficient. This subroutine is called from the main program while the computations are being done for each grid point along the gutter, unless the strip characteristics are identical for which the preceding values can be used. The water quality computations for each subcatchment strip are also performed in this subroutine as will be described in Section VIII. The output from subroutine OVLFLO provides part of the lateral inflow for the gutter routing in the main program.

SUBROUTINE RASNIN: This subroutine computes from rainfall the rate of lateral inflow for the subcatchments and part of the lateral inflow for the gutters. The inflow is evaluated as rainfall plus snowmelt minus infiltration. This subroutine is called from the main program and subroutine OVLFLO.

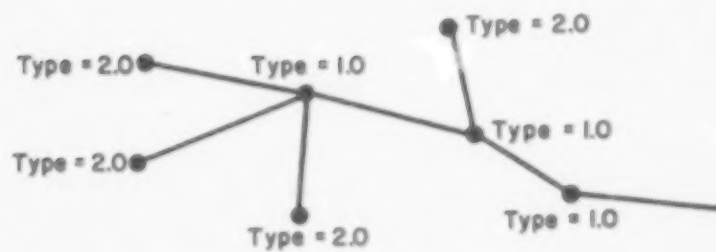
SUBROUTINE UPSBO: This subroutine computes the upstream boundary condition for each gutter. It is called by the main program when computations are made for the upstream end of each gutter at each time level.



(a) Gutter Types



(b) Inlet Types



(c) Sewer Node Types

Fig. 24. Identification of types of gutter and inlet

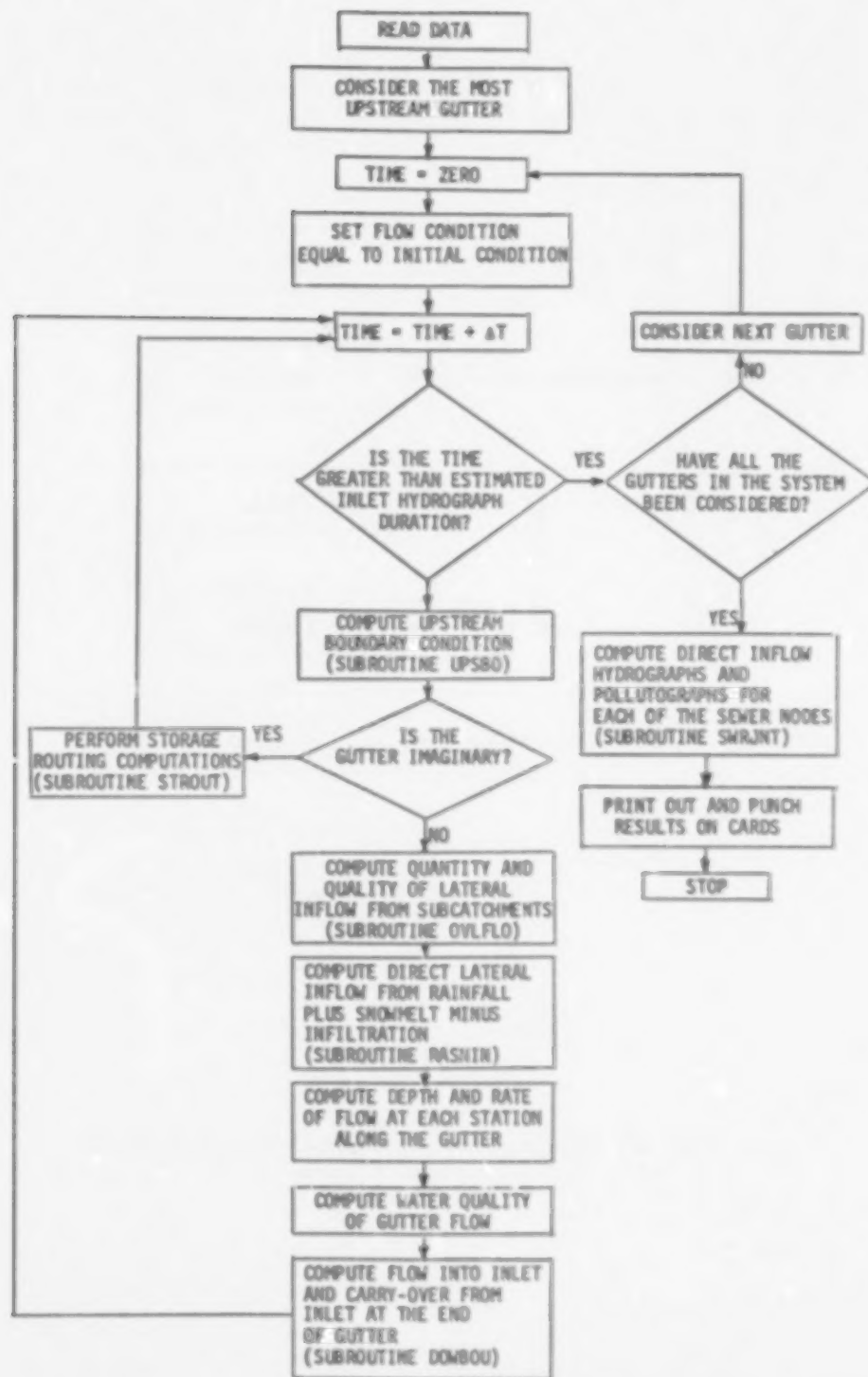


Fig. 25. Flow chart for Illinois Surface Runoff Model computer program

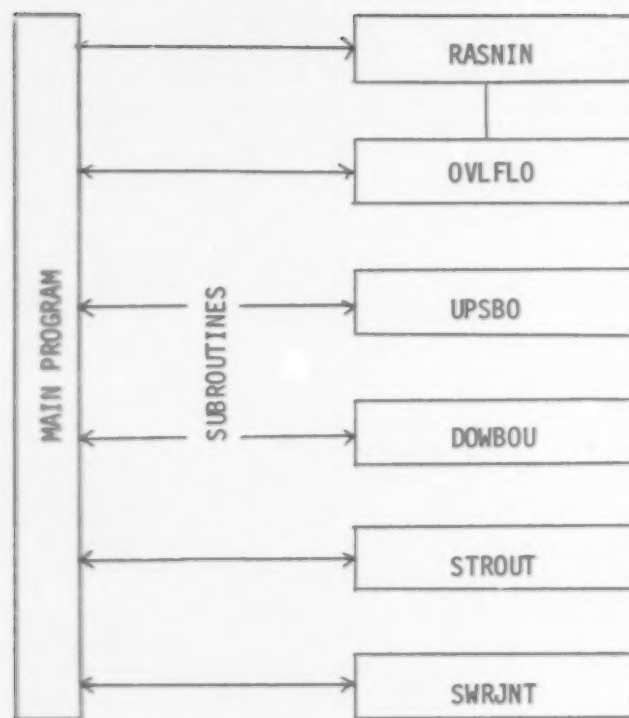


Fig. 26. Composition of Illinois Surface Runoff Model computer program

The carry-over water from all the immediately upstream inlets are summed up and a corresponding flow depth is computed using Manning's formula. This computed depth is accepted as the upstream flow depth for the gutter being examined at the time level being considered. When there is no carry-over, the upstream depth is set equal to the initial depth.

SUBROUTINE DOWBOU: Knowing the gutter outflow as computed by the main program at each time level, this subroutine is called upon to evaluate the inflow into the inlets and the carry-overs. The flow into the inlets is computed by using the weir or orifice formulas (Eqs. 63 and 64) as explained in Sec. VI-3.

SUBROUTINE STROUT: This subroutine provides a storage routing procedure around the inlets where there exist no immediate downstream gutters and hence there is no carry-over. The inflow to the storage area consists of the outflow from the upstream gutters. The outflow from the storage area is the flow into the inlet computed by using the orifice formula (Eq. 64). Whenever a storage routing is required for an inlet, it is necessary to use the concept of imaginary gutter (Type = 0.0 in Fig. 24a) For the imaginary gutter the program does not perform the gutter routing but it calls STROUT to carry out the storage routing.

SUBROUTINE SWRJNT: This subroutine prepares the output (inlet hydrographs) in data cards and provides the link between the Illinois Surface Runoff Model and the Illinois Storm Sewer System Simulation Model. After all the flow hydrographs into the inlets are computed, this subroutine is called by the main program. For the sake of convenience in linking the surface runoff and sewer routing models, the output hydrographs of the surface runoff model are identified by the sewer nodes, i.e., sewer manholes or junctions, instead of the corresponding inlets if they carry different identification numbers. When there are more than one gutter inlets discharging into the same catch basin or sewer node, the ordinates of those inlet hydrographs are summed up. Such a case is identified as Type 2.0 sewer node as shown in Fig. 24c. The computed hydrographs for each of the sewer nodes are provided on computer cards as a part of the surface runoff program output. These cards are in a format compatible to the input data card requirements for the Illinois Storm Sewer System Simulation Model and can be used as part of the data deck for sewer routing. The ordinates of the sewer node inflow hydrographs and pollutographs are also printed out from this subroutine. For the benefit of those who intend to use the Illinois Surface Runoff Model, the instruction on data preparation and listing of the computer program is given in App. B.

SECTION VII

SEWER SYSTEM ROUTING MODEL

As mentioned previously, the sewer routing model of the Illinois Urban Storm Runoff Method is the Illinois Storm Sewer System Simulation Model (ISS Model). The ISS Model actually consists of two options: the design option for which the size of the sewers are to be determined, and the flow prediction option for which the size of the sewers and junctions are known and the objective is to compute the runoff hydrographs for given inputs. Since the ISS Model has been reported in detail elsewhere (Sevuk et al., 1973), and the objective of this research is to investigate methods of prediction for urban storm runoffs for the purposes of pollution control and management, only the flow prediction option of the ISS Model is briefly summarized in this section. Those interested in design of storm sewer networks are recommended to refer to a comparative study on using the ISS and other methods (Yen and Sevuk, 1975).

VII-1. SEWER NETWORK REPRESENTATION

One of the most important aspects in solving sewer flow problems in a network of sewers is to properly and systematically represent the geometric sequences of the sewers. This is particularly important if computer solution is used for which a logical means of selecting the proper order of sewers is a prerequisite of solution. For a small network consisting of a few sewers, it is not difficult to assign specifically the sequence that the computation should follow. For a large system consisting of many sewers, and particularly with the possibility of alternation of the sewer connecting pattern, it is more desirable and practical to set up some rules that the computer can follow to select the sequences. A computer program written for specific sequence can be used only for those networks with patterns following that sequence, and it is necessary to number the sewers precisely as the sequence requires. Contrarily, a computer program written for a certain sequence-selection rule allows arbitrary patterns. In view of the variability of urban sewer systems and the large number of sewers involved in each of the system, the latter approach of setting a rule for sequencing to allow arbitrary numbering of the sewers is adopted in the ISS Model.

In this approach the members of a sewer system are represented by a node-link system commonly used in network analyses. The nodes are the junctions or manholes that join the sewers and each is assigned arbitrarily a number, as shown in Fig. 16 and Table 5 for the Oakdale Avenue Drainage Basin. The sewers are the links in the network and they are represented by two numbers, the first being the number of the upstream node and the second the downstream node. The outlet of sewer system is the root node and is joined by only one sewer. Thus, two sewers that join at the same node each will have one of its two identification

numbers identical to the other's. The sequence becomes a systematic search of marked numbers, and the computer can easily determine the connectivity, i.e., the pattern, of the network.

In solving for the flow in a sewer network, the solution obviously proceeds from upstream sewers toward downstream, no matter if the back-water effect is accounted for or not. To determine which upstream sewers should be solved first and the sequence of the sewers to be solved, a systematic searching method is adopted. The search starts from the root node, i.e., the outlet of the sewer system, to detect if the sewer connected to this node has already been solved. If this connected sewer is not yet solved, then the search moves to its upstream node and the process is repeated again. For nodes joining two or more branches, the process can be proceeded one by one following certain order, e.g., following the relative order of the branches stored in the computer, or following the order, say, from left to right of the branches connected to the node. For example, if the latter rule of left to right is used, the search will start from the root node. Since the flow for the last sewer connecting to the root node has not yet been solved, the search will move to the upstream node of the last sewer. Assuming that there are two other sewers joining to this node, the search will first look if the left one was solved. If it has not been solved, the search will automatically move up to the upstream node of this left sewer and repeat the entire process again. If the left sewer has already been solved, the search will then move to the sewer at the right. If this right sewer has not yet been solved, the search will move to its upstream node. If the right sewer has already been solved, this implies that the sewer with its upstream end joining the node is the only one to be solved. After the solution for this sewer is obtained, the search returns to its downstream end node. In this manner, the solution is obtained systematically, branch by branch, from upstream towards downstream.

VII-2. METHOD OF SOLUTION

In the ISS Model, the flow in each of the sewers is determined by solving the St. Venant equations (Eqs. 42 and 43). The friction slope, S_f , is evaluated by using the Darcy-Weisbach formula (Eq. 45). The Weisbach resistance coefficient f is estimated by using a simplified form of the Moody diagram. Since laminar flow rarely occurs in sewers, only turbulent flow is considered. For fully developed turbulent flow in hydraulically rough conduits, Eq. 50 applies. For hydraulically smooth conduits, the Blasius formula is

$$f = \frac{0.223}{R^{0.25}} \quad (65)$$

for $R < 4 \times 10^5$. In sewers hydraulically smooth boundary flow seldom occurs with $R > 10^6$. Hence if the Blasius formula is assumed to apply up to slightly higher Reynolds number and the transition between smooth-

and rough-surface flows is neglected, the threshold Reynolds number, R^* , is

$$R^* = 0.633 \left(\log \frac{2R}{k} + 0.87 \right)^8 \quad (66)$$

The value of f is computed from Eq. 65 or Eq. 50 depending on whether R is less or greater than R^* .

One initial condition and two boundary conditions are needed to solve the St. Venant equations. For supercritical flow, the two boundary conditions are furnished by the flow conditions at the upstream node of the sewer. This imposes no computational problem since the solution is proceeding towards downstream. However for subcritical flow, one boundary condition is furnished from the upstream node and the other should be from the downstream node. This downstream boundary condition physically represents the backwater effect from the junction to the sewer and usually it is an unknown to be solved. The junction flow condition, in turn, is determined by not only its physical properties but also the flow conditions of all the sewers joining to the junction. Therefore, to solve for the flow in a sewer network, it is necessary either to solve simultaneously all the equations describing mathematically the flows in all the sewers and junctions of the network, or to subdivide the network into components for solution by successive approximations. The first approach is possible and practical for small systems consisting of a few sewers and junctions. For large systems this simultaneous solution method would easily become immanageable. Therefore the second approach is adopted for the ISS Model using a technique called overlapping Y-segments.

In the ISS Model the sewer systems are considered as a tree type network, each consisting of branches formed by a number of connected Y-segments. Each Y-segment contains three sewers joined by a common junction. The hydraulic condition of a junction is accounted for by a dynamic equation in addition to the continuity equation commonly used. The continuity equation is

$$Q_1 + Q_2 + Q_j - Q_3 = \frac{ds}{dt} \quad (67)$$

in which s is the storage in the junction; Q_1 is the direct inflow into the junction; and the subscripts 1 and 2 represent the inflow sewers and 3 the outflow sewer from the junction. If the storage of the junction is negligible, the right-hand side of Eq. 67 is equal to zero.

The dynamic equation for a junction with large storage, i.e., reservoir type junction, is

$$z_1 + h_1 = z_2 + h_2 = z_3 + h_3 + \frac{v_3^2}{2g} \quad (68)$$

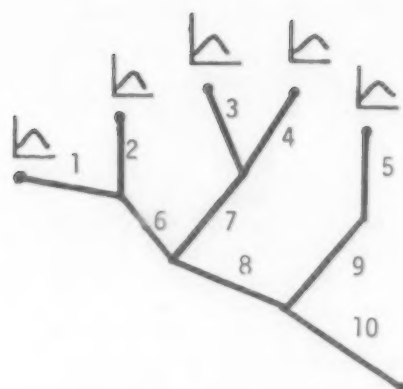
in which h is the depth of sewer flow at the junction and z is the

elevation of the sewer invert above a reference horizontal datum. For a point-type junction with negligible storage, the velocity head term ($V_3^2/2g$) in Eq. 68 is assumed equal to zero. Furthermore, if the inflowing sewer has a drop producing a free-fall of the flow, the depth for that sewer is equal to the critical flow depth corresponding to the instantaneous discharge of the sewer.

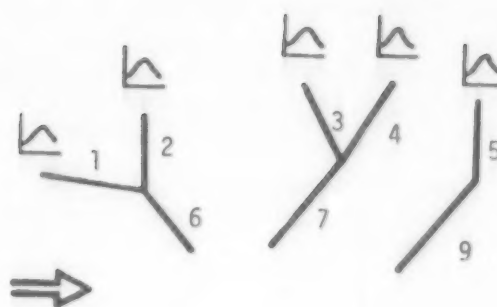
The St. Venant equations (Eqs. 42 and 43) and the junction equations (Eqs. 67 and 68) can be solved simultaneously for a Y-segment with known upstream boundary conditions of the inflowing sewers and assumed (if unknown) downstream boundary condition for the outflowing sewer. Since the downstream boundary condition is assumed, the solution is only approximate and a successive overlapping Y-segment technique is adopted to improve the accuracy of the solution. The technique is shown schematically in Fig. 27. Numerical solutions are first obtained, branch by branch, for those Y-segments whose inflowing sewers are connected to the inlet catch basins. The prescribed inlet hydrographs (or the outflow hydrographs from the Illinois Surface Runoff Model) and the compatibility conditions at the junction are used as the boundary conditions. In addition, if the downstream condition of the Y-segment is unknown, the forward differences are used as a substitution for the downstream boundary condition. The solution is obtained by applying the St. Venant equations to each of the three sewers and Eqs. 67 and 68 to the junction and solving these equations simultaneously for the Y-segment using a first-order characteristic method (Sevuk et al., 1973). After the computation is completed, the first trial solution for the outflowing sewer is discarded but the "true" solution for the inflowing sewers is retained. Thus the inflow hydrograph into the junction of the current Y-segment is obtained. This junction will serve as one of the two inlets of the next Y-segment and the original outflowing sewer will become an inflowing sewer for the advanced new Y-segment. This procedure is repeated until the entire network is solved. For the last segment of the system, the prescribed boundary condition at its downstream end, the outlet of the system, is used and thus the numerical solution over the entire network is completed.

The ISS Model, in its present form, can simulate the flow with regulating and operational devices if they are located at the system outlet. From the hydraulics viewpoint, such control facilities can be expressed mathematically as stage-time relationship $h = f(t)$; velocity-depth relationship $V = f(h)$; discharge-depth relationship $Q = f(h)$; and discharge-time relationship $Q = f(t)$. Consideration of such control facilities not only provides a means to account for the many different types of regulators often used in storm sewer systems but also provides a mean to decompose large sewer systems into smaller sequential subsystem for easier management and analysis.

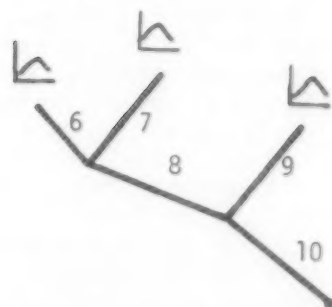
Similar to the case of solving numerically the overland and gutter flows, the initial condition for the sewer routing cannot be dry-bed; i.e., zero depth and zero velocity will impose a computational singularity. For



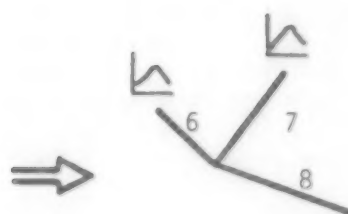
a. Complete solution domain



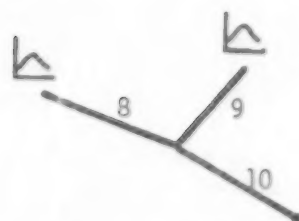
b. Solution domains for first-order sewers (1 through 5)



c. Reduced solution domain



d. Solution domain for second-order sewers (6 and 7)



e. Final solution domain for the sewers 8, 9, and 10

Fig. 27. Solution by method of overlapping y-segments

combined sewers, the initial condition can be evaluated from dry-weather flow. For storm sewers without initial base flow, a small but negligible base flow is assumed to start the computation.

In using the overlapping Y-segment technique, only three sewers are considered at a junction. For a junction with more than three joining sewers, only three can be considered for direct backwater effects. Others, preferably those with small backwater effects from the junction, can be treated as direct inflows, i.e., as Q_1 in Eq. 67. For a junction joined by only two sewers, the third sewer of the Y-segment can be considered as imaginary with zero length.

VII-3. COMPUTER PROGRAM DESCRIPTION

The ISS Model just described has been programmed for computer solution. A macro flow chart showing the logic of solution for the flow prediction option of the computer program is given in Fig. 28. The program begins its execution by reading a set of control and data cards which describes the run control specifications (user commands), sewer system layout and physical characteristics of system components and inflow hydrographs. After verifying the accuracy and completeness of this information, the program starts to execute the numerical simulation phase. Output from the numerical simulation consists of a tabular print out of the computed or specified sewer diameters, time variations of flow rate, velocity, and depth at sewer entrances, and space variations of flow rate, velocity, and depth along the sewers at specific time intervals. In addition to this default print out (the amount of which is under the control of user through various control card options), optional data capture facilities are provided which allow the user to produce his own plots or graphic displays of flow behavior at the inlets, junctions, and outlets of the network. After completion of the flow simulation for one sewer network, the program proceeds on to the next network, if any, until all such networks are processed.

At its present form, the computer program of the ISS Model cannot account for moving hydraulic jumps and surges within the sewers and only circular sewers can be considered. The latter restriction may be removed, since for sewers with noncircular cross sections, simulation can still be made by using equivalent diameters of the conduits based on best fit of hydraulic radius or cross sectional area.

The computer program of the ISS Model consists of around 3,000 statements written in PL/I-source code, as well as an assembler language subroutine. The PL/I portion includes several short external subroutines, a main controlling section with network control card input routines, and a large set of numerical computation routines. The program is written to be composed of modules, whenever possible, so that most of the routines are separately compiled and then linkage edited together with the assembler language routine to form the executable load module.

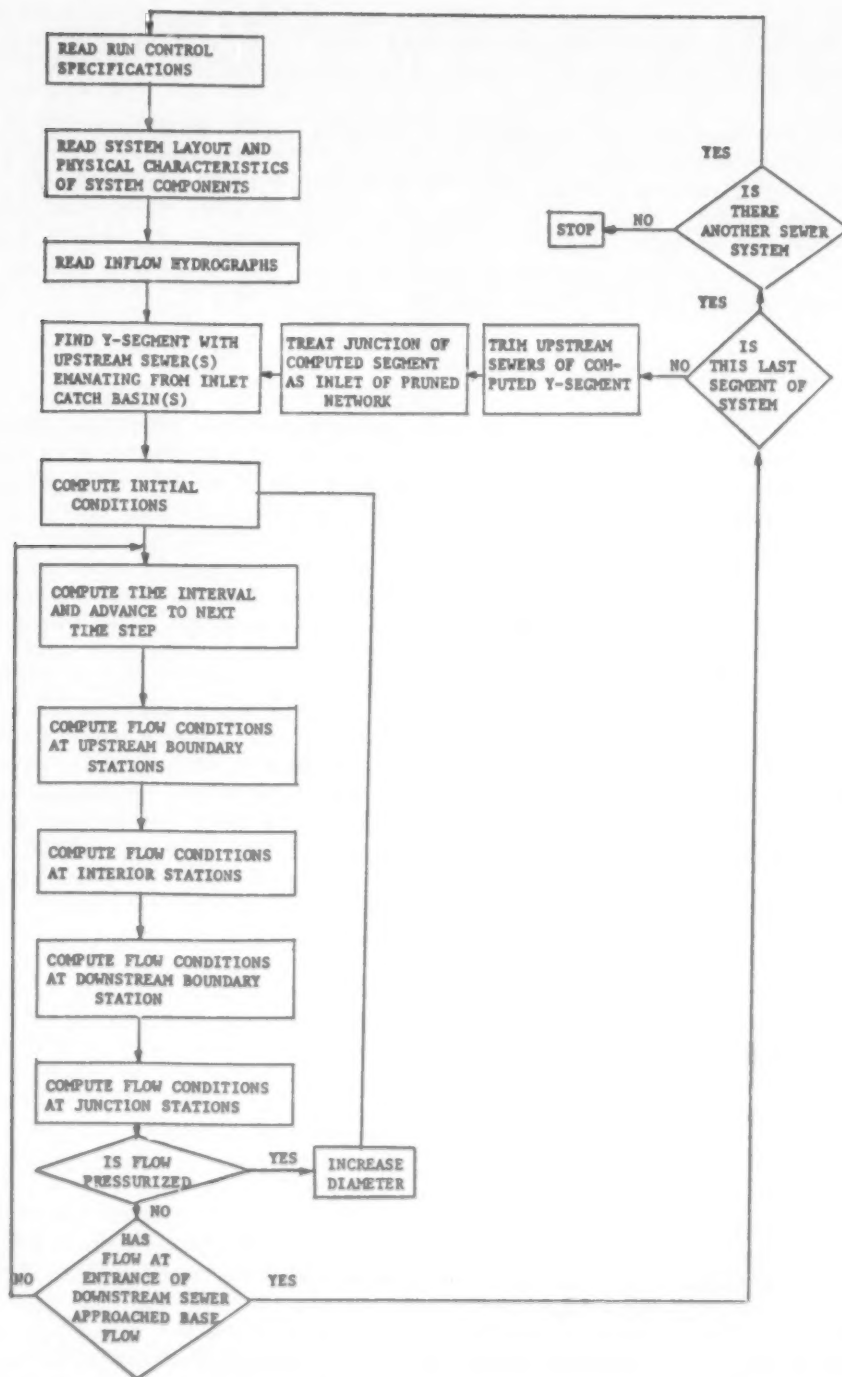


Fig. 28. Flow chart for ISS model computer program flow prediction option

The program can be adapted for execution on most large IBM 360 or 370 models running under OS/360 (preferably OS/MVT) operating system or its VS equivalent. A memory storage of 300K bytes is desirable although under certain conditions 100K bytes may be acceptable. The machine must have the floating point instruction set. In implementation, the program resides on a direct-access storage device, such as a disk, a drum, or a data cell drive. This enables the program to be loaded quickly and efficiently into the main storage without entailing the added overhead of recompilation each time the program is executed.

Despite the sophistications of the theory and programming techniques of the ISS Model, the computer program was written for easy adoption by users having only elementary computer knowledge. Programming experiences in PL/1 language will be helpful but not essential for the implementation of the program. The program can be implemented through the use of either a distribution tape or the program listing. The former approach is recommended, particularly for those users who are not technically oriented on computer operations. Prospective users may obtain the distribution tapes at cost of the magnetic tape, handling, postage, and computer time for duplication.

Standard distribution tapes are provided on 1600 bytes per inch, 2400 ft magnetic tapes. When requesting for a distribution tape the following information should be provided:

- (a) Model of computer system, e.g., IBM System 370, model 168 MP.
- (b) Operating system in use, e.g., OS/MVT with HASP.
- (c) Region size available for the program, e.g., 220K.
- (d) Type of disk or other direct-access device to which the program will be transferred from the distribution tape, e.g., 2314, 2319, 3330.
- (e) Compilers available at the installation, e.g., either, neither or both of the OS PL/1 (F) and PL/1 Optimizing compilers.

However, users who prefer to use the program listing for implementation or modification should have adequate understanding of the following IBM systems 360 and 370 concepts:

- (a) creation, use, and modification of partitioned data sets for storage of source and object decks, as well as load modules;
- (b) use of the linkage editor to produce executable load modules, and method of executing the load modules thus created;

- (c) use of other IBM system utilities, such as IEHMOVE, IEBUPDTE, etc;
- (d) use of tapes, disks, and other direct access devices; and
- (e) use of IBM job control language (commonly referred to as JCL).

Those who need to use the program listing and yet unfamiliar with computer applications are advised to seek assistance from experienced programmers.

The listing of the computer program, input and output data format, program structures, operational procedure and the finite differences equations used for numerical solution have been reported in details by Sevuk et al. (1973) and are not repeated here for brevity. Those interested in details should refer to that report.

SECTION VIII

WATER QUALITY MODEL

As mentioned in Section III, Introduction, storm runoff has been known as a significant source of pollution because of its capability to wash and carry pollutants on its course of flow. With recent stringent requirements on water quality for the control of pollution, the necessity, desirability, and economical feasibility of treatment of storm runoff is often an unavoidable concern. For combined sewer systems, the problem is further complicated by the variable quality as well as the quantity of the dry weather flow.

For pollution control purposes, ideally the time and spatial distributions of the quality of runoff should be known, at least at certain key locations like sewer outlets and overflows. This knowledge is particularly useful, for example, for a selective withdraw and treatment of storm runoff. However, such detailed information requires, from an experimental viewpoint, the time-consuming, tedious and expensive measurements and laborious analyses of the data, and from a theoretical viewpoint, the precise theories concerning diffusion and dispersion of pollutants and the chemical, biological, and thermal processes involved. Furthermore, a prerequisite for an accurate water quality analysis is a reliable quantitative prediction of water.

The water quality model introduced in this section is an attempt to provide a means to evaluate water quality of storm runoff as a supplement to the quantitative evaluations of the Illinois Surface Runoff Model and the ISS Model. The model is a relatively simple one by using a one-dimensional approach considering the time and spatial variations of the pollutant expressed in terms of cross-sectional averaged concentration. Only the equation of conservation of mass is used. Implicitly this involves the assumption of instant mixing within the flow cross section. The dynamic equation expressing the gravity effect (buoyancy or settling) is not used in the model. Neither are the effects of biological and chemical reactions accounted for. Improvements on these aspects, of course, can and should be made in the future. However, this is to be done after the simple model has been adequately tested with data. At present available field measurements are mostly for a limited number of locations in a drainage basin and do not provide enough data on spatial and temporal variations of storm runoff water quality to verify the model sufficiently.

VIII-1. WATER QUALITY MODEL FORMULATION

The water quality model described here utilizes the computed discharge hydrographs and flow area-(or depth-) time relationship at desired locations of the drainage system from the surface runoff and ISS Models together with the initial pollutant distribution to evaluate the

transport of pollutant by storm runoff. The output is a set of polluto-graphs (pollutant concentration-time graphs) at the desired locations.

The mass conservation equation of a pollutant expressed in concentration c for the flow with a discharge Q and flow cross sectional area A is

$$\frac{\partial cA}{\partial t} + \frac{\partial cQ}{\partial x} = c_\ell q_\ell \quad (69)$$

in which c_ℓ is the concentration for the lateral flow q_ℓ . The term $c_\ell q_\ell$ includes concentrated pollutant dosage in which case the value of $c_\ell q_\ell$ is simply replaced by the value of the dosage. Substitution of the continuity equation (Eq. 52) into Eq. 69 yields

$$A \frac{\partial c}{\partial t} + Q \frac{\partial c}{\partial x} = q_\ell (c_\ell - c) \quad (70)$$

By using the computational grid shown in Fig. 23, Eq. 70 can be written in finite difference form as

$$A_C \frac{c_C - c_B}{\Delta t} + Q_C \frac{c_C - c_D}{\Delta x} = q_\ell (c_\ell - c) \quad (71)$$

However, because of the details of the output of the Illinois Surface Runoff Model and the ISS Model on runoff quantities, the computational procedure for water quality for the surface runoff and the sewer runoff are different. For the surface runoff part, the water quality computation is programmed in the Illinois Surface Runoff Model and the quantity and quality computations are done concurrently. In the sewer system runoff part, the water quality model is programmed separately using the output from the ISS Model as the input.

(A) Water quality computation for surface runoff

Water quality for surface runoff is computed within the Illinois Surface Runoff Model. In Eq. 71, if the computational grid points D and C represent respectively the upstream and downstream ends of a gutter or subcatchment strip, the flow parameters for D at each time level are known from the upstream boundary conditions. The values of A_C and Q_C are supplied from the water quantity computation of the model. For gutter flows, the values of c_ℓ and q_ℓ are known from the subcatchment runoff. For subcatchment flows, the values of c_ℓ and q_ℓ are known from rainfall (usually with $c_\ell = 0$) and other known lateral flows, if any. Hence, from Eq. 71 the concentration c_C can be solved explicitly for each time level as

$$c_C = \frac{q_\ell c_\ell + \frac{c_B A_C}{\Delta t} + \frac{c_D Q_C}{\Delta x}}{\frac{A_C}{\Delta t} + \frac{Q_C}{\Delta x} + q_\ell} \quad (72)$$

(B) Water quality model for sewer system runoff

In view of water quality routing, a sewer system can be considered to consist of two different types of elements; namely, the sewer conduits and the junctions with substantial storage capacities. For a sewer with no lateral flow, Eq. 71 yields for the downstream end C

$$\left(\frac{A_C}{\Delta t} + \frac{Q_C}{\Delta x}\right) c_C - \frac{Q_C}{\Delta x} c_D = \frac{c_B A_C}{\Delta t} \quad (73)$$

and for the upstream end

$$\frac{Q_D}{\Delta x} c_C + \left(\frac{A_D}{\Delta t} - \frac{Q_D}{\Delta x}\right) c_D = \frac{c_A A_D}{\Delta t} \quad (74)$$

With the discharge and flow cross-sectional area given from the output of the ISS Model, and c_B and c_A known from the initial condition or previous time computations, Eqs. 73 and 74 can be solved for c_C and c_D at each time level using Cramer's rule. This procedure provides the pollutographs at both ends of sewers which can also be used for water quality computation at sewer junctions.

For a sewer junction, conservation of mass of the pollutant gives

$$\frac{d(cs)}{dt} = \sum_i c_i Q_i + \sum_j c_j Q_j \quad (75)$$

in which c is the pollutant concentration for the volume of water s in the junction; c_i is the concentration of discharge Q_i from the i -th sewer into the junction, Q being positive for inflow and negative for outflow; and c_j is the concentration for the j -th direct inflow Q_j . Writing Eq. 75 in finite difference form and solving for the concentration at the present time level one obtains

$$c_p = \frac{1}{h_p} \left[c_o h_o + \frac{\Delta t}{A} \left(\sum_i c_i Q_i + \sum_j c_j Q_j \right) \right] \quad (76)$$

in which A is the constant horizontal cross sectional area of the junction being considered; h is the depth of water in the junction; and the subscripts (p) and (o) denote respectively the present and previous time levels.

VIII-2. COMPUTER PROGRAM DESCRIPTION FOR SURFACE RUNOFF QUALITY

As mentioned previously, the water quality computations for the surface runoff and sewer system are handled separately. The computation for the surface runoff quality is done as a part of the Illinois Surface Runoff

Model whereas that for the sewer is done through a program supplement to the ISS Model. Because the discharges and flow cross-sectional areas of the gutter and subcatchment flows required by Eq. 72 for quality computation are computed in the surface runoff model but not provided as a part of the surface runoff output, it is more advantageous to integrate the quality computation into the surface runoff program than to separate it. Since the Illinois Surface Runoff Model Program has been described in Sec. VI-4, only the quality part of the program is described here.

The computer program for the surface runoff allows the consideration of two pollutants at a time. The quality of subcatchment flow is computed in subroutine OVLFO using Eq. 71. At the upstream end of a subcatchment strip, the concentration of each pollutant is assumed either to remain the same as the initial concentration or to vary according to a prescribed function of time. Knowing the flow from the subcatchments, the water quality computations for the gutter flow are made in the MAIN PROGRAM. In Eq. 71, the total lateral flow into a gutter from all the components (subcatchments, direct rainfall, etc.) is computed at every time level and averaged over the length of the gutter. The concentration in lateral inflow is evaluated as the average of component flow concentrations weighted with respect to flow rate. The upstream boundary condition for a gutter is computed in subroutine UPSBO. The concentration of each pollutant at the upstream end of the gutter is evaluated at each time level as the average of concentrations weighted over the flow rate of the carry-overs from the immediately upstream inlets. When there is no upstream inlets, the concentration at the upstream end is equal to the initial value at all time levels of computation. The quality of flow into an inlet is assumed to be the same as the corresponding gutter outflow. When there are more than one gutter inlets discharging into the same catch basin or sewer node, an average pollutograph for each pollutant weighted with respect to the inlet flow rates is computed for the sewer node in subroutine SWRJNT.

The data required for the quality part of the surface runoff model consists of the initial concentration of each pollutant in subcatchment strips and gutters. The input format and the preparation of data cards will be described in App. B. The output from the computer program includes the ordinates of direct inflow hydrographs and the corresponding pollutographs for all the sewer nodes printed out at equal time intervals.

VIII-3. COMPUTER PROGRAM DESCRIPTION FOR ILLINOIS SEWER SYSTEM WATER QUALITY MODEL

The sewer system water quality model is programmed in Fortran IV language. The input to the computer program includes the data on depths and discharges at the entrance and exit of each sewer and the volume of water at each storage junction at given times as provided by the output of the ISS Model. In addition, the pollutographs as obtained from the surface runoff quality computation, and the direct inflow hydrographs,

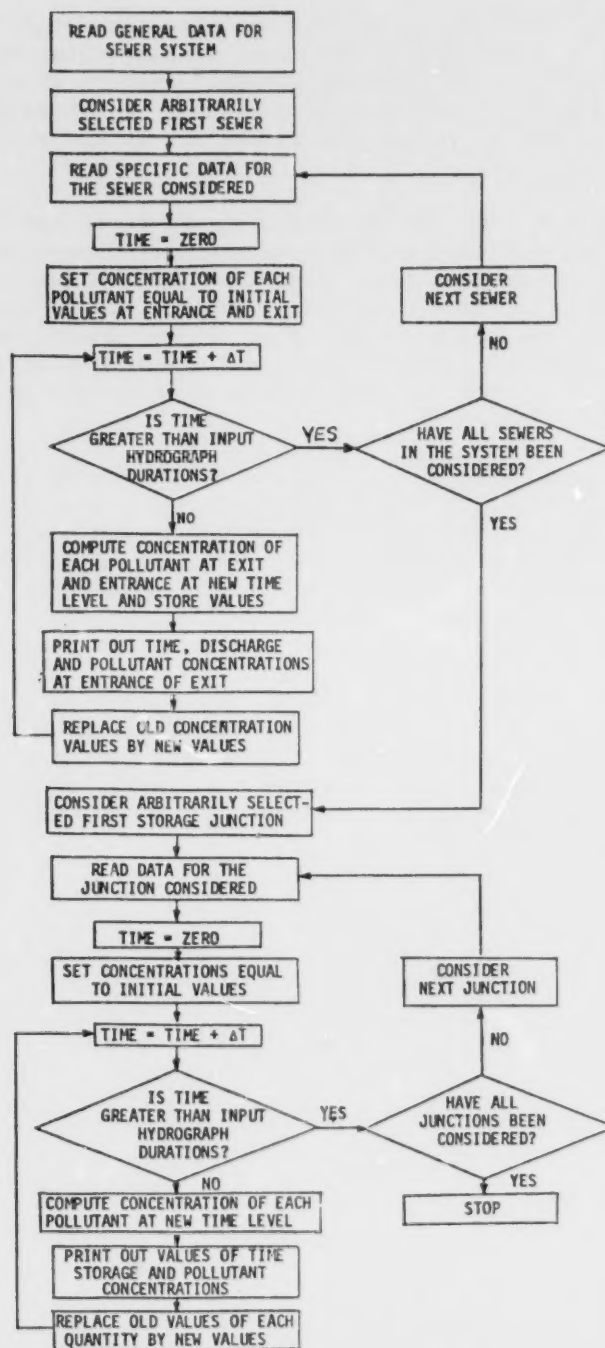


Fig. 29. Flow chart for Illinois Sewer System Water Quality Model computer program

if any, and the sewer system layout are also input into the program for runoff quality routing in the sewer system, using Eqs. 73, 74, and 76. The output from the computer program consists of the pollutographs representing the time variation of pollutant concentration at the sewer system outlet, at the storage junctions, and at the entrance and exit of all sewers.

Because the output from the ISS Model on the depth and discharge at the entrance and exit of each of the sewers are at irregular time intervals, the data of the sewer hydrographs as input to the quality model need not be provided at equal time intervals. However, the direct inflow into the sewer junctions and the water stage in the junctions can be obtained respectively from the surface runoff and sewer routing models at constant time intervals. Therefore these values are provided as input at regular time intervals.

The computer program of the Illinois Sewer System Water Quality Model allows the consideration of two different pollutants at a time. The sewer system may consist of as many as 100 sewers. Arbitrary identification numbers can be assigned to the sewer nodes (junctions and manholes), although preferably for simplicity the same numbers as the ISS Model should be used. Because the backwater effect is already accounted for in the quantity computation, the sewer system is not restricted to tree-type networks. The computer program consists of approximately 200 statements and the storage requirement is 300 K. When a sewer system consisting of more than 100 sewers is to be considered, the program should be modified by simply changing the DIMENSION statements. This may cause an increase in storage requirement. The computer program is listed in App. C together with information on data preparation, and the computational logic is shown schematically in Fig. 29.

SECTION IX

GENERAL DESCRIPTION OF METHODS EVALUATED

Numerous rainfall-runoff "models" have been proposed by previous investigators. Most of the early models were developed for rural areas. These include the Burkli-Ziegler and other similar formulas (Chow, 1962), the monograph methods such as the methods of ARS-SCS (U.S. Soil Conservation Service, 1971), BPR (Potter, 1961), California (1953), Chow (1962), and Cook (Hamilton and Jepson, 1940), and many of the hydrograph methods.

Among those models applicable to urban areas, some are strictly for overland runoff prediction. These include Izzard's (1946) and Horton's (1938) methods. The models which consider both overland and sewer flows can be divided into two groups. The lumped system group, including the rational method and the unit hydrograph method, treats the drainage basin as a black box producing output (basin runoff) from given input without considering what is happening to the flowing water within the basin. The distributed system approach which includes most other urban runoff models, routes the rainfall excess through the overland surface and sewers to produce the runoff hydrographs.

It would be tedious to list and costly to compare in this study all the methods applicable to urban drainage systems. Therefore, only eight methods which are either most well known and widely adopted or having great application potentials are evaluated in this report. These methods as they appeared in fall 1974 are briefly described in this section, following roughly their relative order-of-complexity. Major features of these eight methods are listed in Table 6. Presumably the most authoritative description of the procedures in using a method is that by the original method developer. Therefore, the steps for application of these methods are not repeated here. Those who want to use the particular methods should refer to the original reports for detailed procedures.

IX-1. THE RATIONAL METHOD

The rational method is the oldest, simplest, and most widely adopted method for storm runoff estimation (Chow, 1962). In the rational method, the peak rate of storm runoff, Q_p , is estimated as

$$Q_p = CiA \quad (91)$$

in which C is a dimensionless runoff coefficient; i is the rainfall intensity and A is the size of the drainage area. The infiltration and other abstractions from the rainfall is implicitly accounted for by the runoff coefficient. The value of i is equal to the average rainfall intensity over a duration equal to the so-called time of concentration.

Table 6. URBAN RUNOFF PREDICTION METHODS EVALUATED

Model	Input data		Basin properties	Runoff routing			Output results	Selected references
	Rainfall	Abstractions		Overland	Gutters	Sewers		
National Unit hydrograph	Average intensity over the duration Hyetograph	Accounted by runoff coefficient Infiltration by ϕ index or Horton's formula; other abstractions, if accounttable	Basin size Base flow	—	—	—	Peak discharge Basin runoff hydrograph	Chow, 1959 ASCE, 1964 Chow, 1964
Chicago hydrograph	Hyetograph	Infiltration by Horton's formula and depression storage by an exponential function	Overland surfaces: length, slope, cross-sectional dimensions and roughness of gutters and sewers	Izzard's method	Linear kinematic wave storage routing with Manning's formula	Linear kinematic wave storage routing with Manning's formula or time off, t method	Basin runoff hydrograph	Tholin and Keifer, 1960
TRAL	Hyetograph	Pervious areas produce no runoff and all rainfall on impervious areas becomes runoff	Areas of directly contributing impervious surfaces; time of travel of impervious areas	Flow time - area method	—	Reservoir routing, lagged time of travel, t	Basin runoff hydrograph	Watkins, 1962; Terstriep and Stall, 1969
UCUR	Hyetograph	Infiltration from rainfall only by Horton's formula and depression storage by an exponential function	Length, slope, and n for overland surfaces; length of gutters; diameter, slope and n of sewers	Manning's formula and empirical detention storage function	Continuity equation of steady spatially varied flow	No routing, lagged by time of travel in sewers	Runoff hydrograph	Papadakis and Preul, 1972; Univ. of Cincinnati, 1970

Table 6 (continued). URBAN RUNOFF PREDICTION METHODS EVALUATED

Model	Input data		Runoff routing			Output results	Selected references
	Rainfall	Abstractions	Basin properties	Overland	Gutters	Sewers	
Dorsch	Hyetographs, allows areal variation	Infiltration for subcatchments by Horton's formula and depression storage age	Overland surface length, slope, roughness and depression; sewer size, length, slope and roughness	Linear kinematic wave routing with uniform depth continuity equation and Manning's formula	Linear kinematic wave model, storage routing with Manning's formula and continuity equation	Diffusion-wave eq. with partial backwater effects	Klym et al., 1972 Vogel and Klym, 1973
EPA SWMM	Hyetographs, allows areal variation	Infiltration for subcatchments by Horton's formula and depression storage age values	Overland surface length, width, roughness n, slope and percent imperviousness; length, slope, cross-sectional dimensions and roughness of gutters and sewers	Linear kinematic wave model, storage routing with uniform depth continuity equation and Manning's formula	Linear kinematic wave model, storage routing with Manning's formula and continuity equation	Improved non-linear kinematic wave model with partial backwater effects	Metcalf & Eddy, Inc. et al., 1971 Heaney et al., 1973 Huber et al., 1975
Illinois	Hyetographs, allows areal variation	Infiltration for subcatchments by Horton's formula and initial detention storage age	Length, width, slope, and roughness of overland surface elements; length, roughness, cross-sectional dimensions and slope of gutters; type and dimensions of inlets; length, slope, roughness and diameter of sewers, size of manholes and junctions	Nonlinear kinematic wave with Darcy-Weisbach's formula	Nonlinear kinematic wave with Manning's formula	St. Venant eqs. with backwater effects	Sevuk et al., 1973 Sections VI, VII and VIII

Details of application of the rational method to urban storm sewer design can be found elsewhere (e.g., ASCE and WPCF, 1969; Yen et al., 1974). A block diagram showing schematically the use of the rational method for storm runoff peak discharge estimation is given in Fig. 30.

The drawbacks of the rational method have been discussed by many investigators (e.g., see Chow, 1964; McPherson, 1969). For urban storm runoff quantity and quality control, the most serious drawback of the rational formula is that it gives only the peak discharge, Q_p , and provides no information on the time distribution of the storm runoff.

IX-2. UNIT HYDROGRAPH METHOD

Since Sherman (1932) proposed the concept of unit hydrograph, it has been used to study the rainfall-runoff relationship for rural as well as for urban areas. A unit hydrograph for a drainage basin is defined as the discharge-time graph (hydrograph) of a unit volume of direct runoff (usually expressed as unit depth) from the basin produced by an areally and temporally uniformly distributed effective rainfall of a specified unit duration. The unit hydrograph for a drainage basin is obtained by reduction from previous rainfall and runoff data, by synthetic means, or by transpose of the unit hydrograph from a neighboring basin of similar physical characteristics. The unit hydrographs for different durations can be obtained by direct derivation or by the S-hydrograph method. The procedures to derive the unit hydrograph for a drainage area and to apply it can be found in standard hydrology reference books (e.g., Chow, 1964).

With the unit hydrographs of different durations for a given drainage basin known, the procedure to produce runoff hydrographs for given rainstorms is as follows:

- (a) The rainfall excess is first computed by subtracting abstractions from the total rainfall. For urban storm runoff studies among the different abstractions usually only infiltration is considered although other abstractions can also be included without causing much difficulty. Often the ϕ -index method of constant infiltration rate is used because of its simplicity to estimate the infiltration, although Horton's formula (Eq. 34) and other methods can also be used.
- (b) Subdivide the rainfall excess obtained in (a) into a number of small rainstorms of various durations each of which has nearly uniform distribution of intensity of rainfall excess over its duration. Determine the amount (depth) of rainfall excess, duration, and the beginning time for each of these subdivided rainstorms.

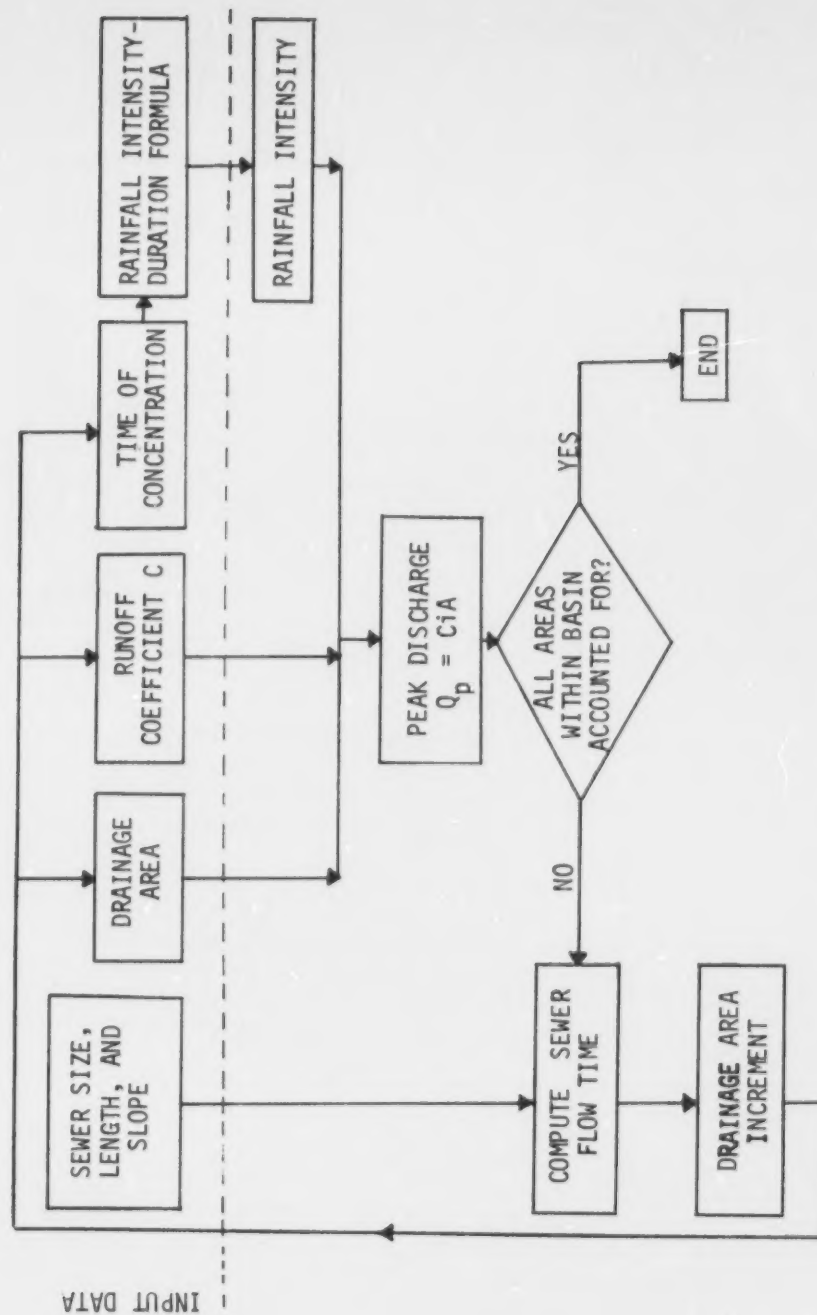


Fig. 30. Schematic diagram of the rational method for peak discharge prediction

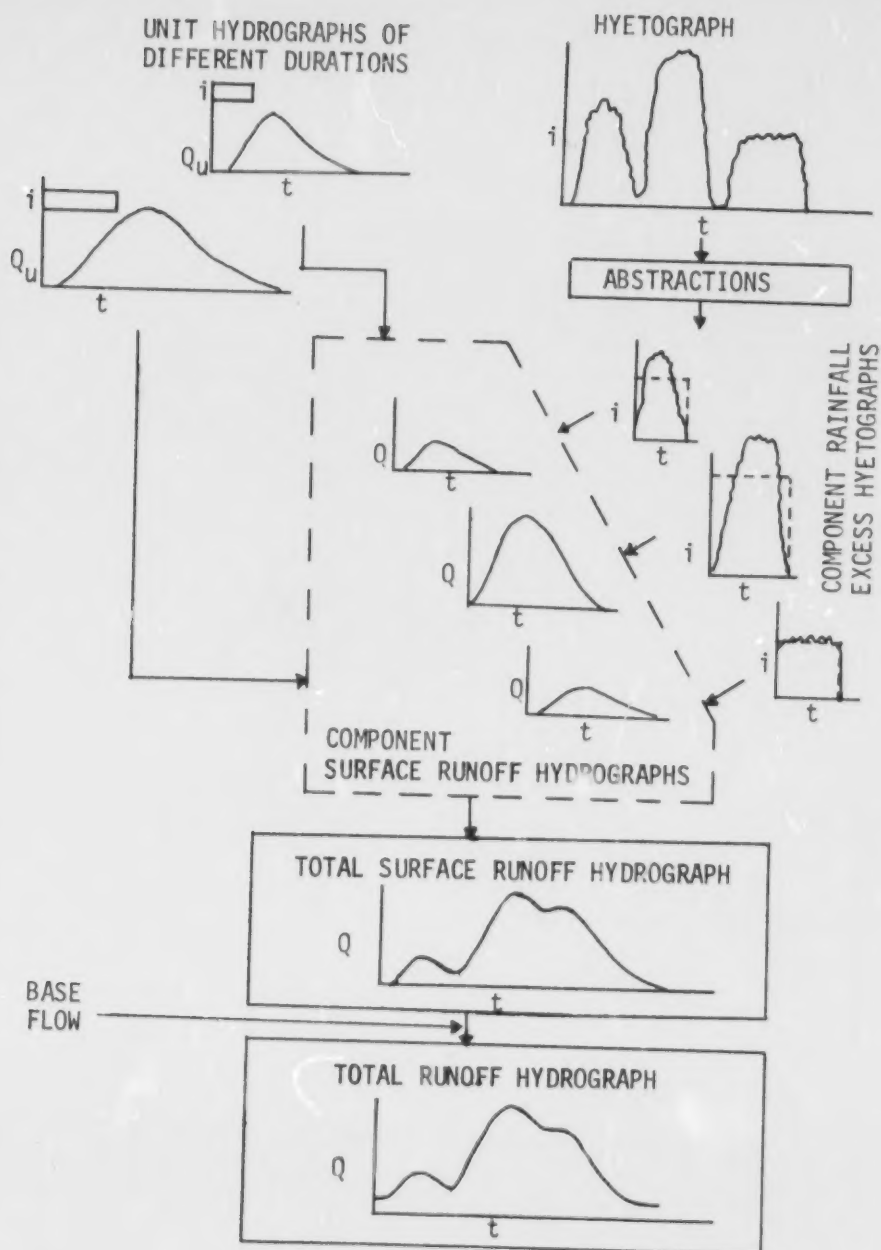


Fig. 31. Schematic diagram of unit hydrograph method

- (c) For each of these subdivided rainstorms, apply the unit hydrograph with a duration closest to the rainstorm's duration. The ordinates of the unit hydrograph are multiplied by the depth of the rainfall excess to give the runoff hydrograph for that component rainstorm. This procedure is repeated in sequence for all the rainfall excesses of different depths and durations occurring at different times. The resulted runoff hydrographs for all the component rainstorms are then added linearly with appropriate time shifting (to account for the different times for different rainfall excesses) to give the combined runoff hydrograph due to the rainstorm.
- (d) A base flow, such as dry weather flow, is added to the combined runoff hydrograph obtained in (c) to give the total runoff hydrograph.

A block diagram of the unit hydrograph method is shown in Fig. 31.

For urban drainage basins which are often of the size smaller than several km^2 (several sq mi), reliable unit hydrographs usually cannot be established because of lack of data. Also, for the unit hydrographs obtained based on past records to be applicable, the drainage basin characteristics must be time invariant, i.e., no significant changes of the physical properties of the basin in time. Furthermore, from the fluid mechanics viewpoint the unit hydrograph theory may not be reliable for small drainage areas of a few acres (hectares) or smaller.

IX-3. CHICAGO HYDROGRAPH METHOD

The Chicago method is a steady-flow hydrograph routing method to determine the time distribution of the quantity of storm runoff (Tholin and Keifer, 1960). The method takes into consideration storages in gutters and sewers. For a given drainage area and rainstorm, the infiltration is computed by using Horton's formula (Eq. 34) and the surface depression storage is computed by using an empirical function (Eq. 35). The computed rainfall excess is then routed through the overland surface using a modified Izzard's method. The gutter flow is routed using a storage routing method with Manning's formula. The same routing method is applied to the sewer laterals and mains. To simplify the computation, a time-offset method to subdivide the hydrographs for sewer routing was also proposed. However, this time-offset method has no theoretical basis and is not used in the present study. A block diagram showing the structure of the Chicago method is given in Fig. 32.

Detail procedure of the Chicago hydrograph method is described in a paper by Tholin and Keifer (1960). Hand computation of the method is time consuming. The Department of Public Works of the City of Chicago has a computer program of the Chicago method.

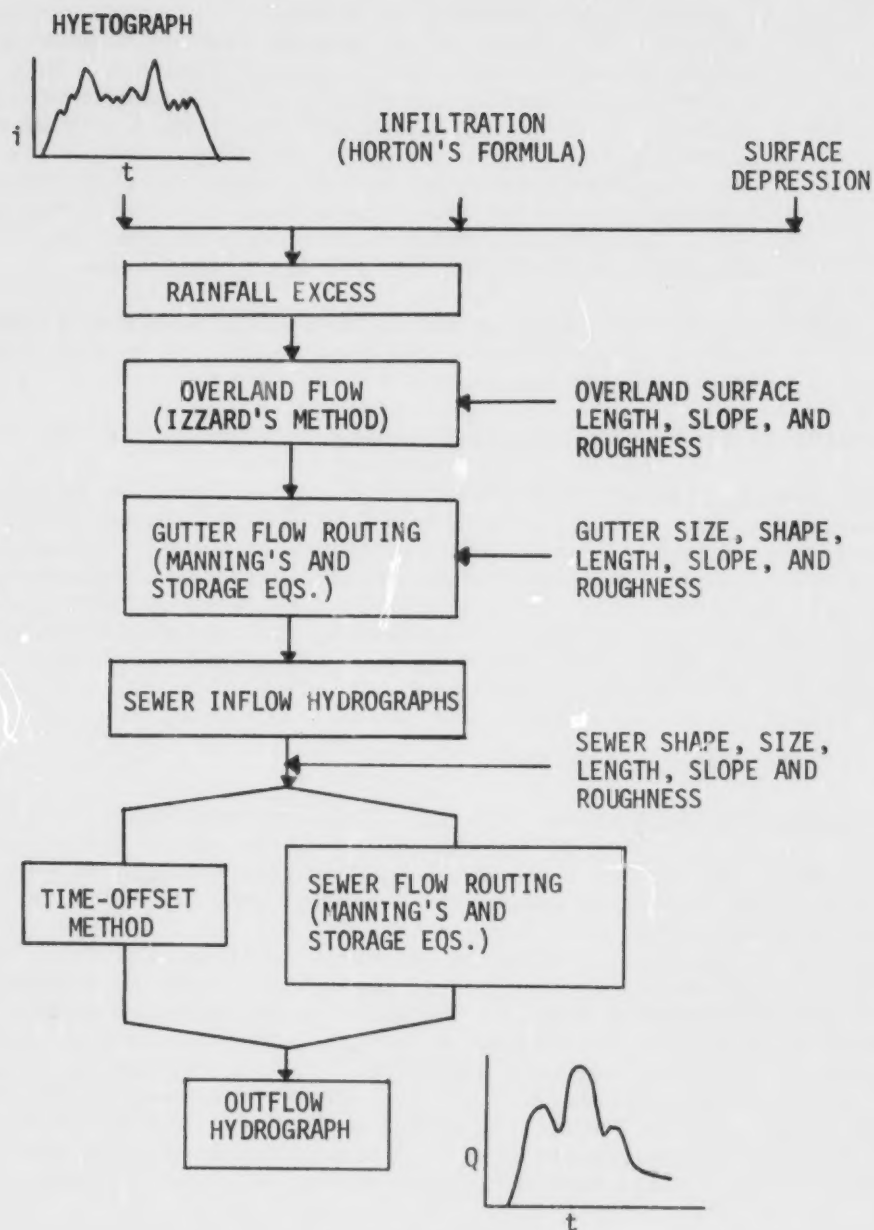


Fig. 32. Schematic diagram of Chicago hydrograph method

IX-4. TRANSPORT AND ROAD RESEARCH LABORATORY METHOD

The British Transport and Road Research Laboratory (TRRL) method is another hydrograph routing method developed specifically for urban areas (Watkins, 1962; Terstriep and Stall, 1969). In the United States it is still generally known by its original name, RRL method. The method was developed for the purpose of determination of design runoff hydrographs although it can also be used for flow prediction purpose. The method assumes that pervious areas and impervious areas not directly connected to the drainage system produce no runoff, and all the rainfall on impervious areas directly connected to the drainage system becomes runoff. A linear flow time-area concept similar to that adopted in the development of Chow's method (1962) is used to establish the hydrographs for the impervious areas. A time of entry, similar to the time of concentration for the rational method, is estimated by experience as the time required for the directly connected impervious area to contribute to the flow into the inlet catch basin. Terstriep and Stall (1969) proposed to use a formula suggested by Hicks to compute the time of entry for overland flow and Izzard's modification of Manning's formula for gutter flow. Thus, in the original version the required overland and gutter data consist only of the area of the directly connected impervious subcatchments and their time of entry, whereas for the Terstriep and Stall version the slope and length of the overland surfaces and the slope, shape, and Manning's n for the gutters are required. The inflows from different contributing areas of an inlet are combined linearly with appropriate time lag to give the inlet inflow hydrograph.

The inlet hydrographs are then routed through the sewers using a reservoir routing technique. The time of travel in a sewer is computed as $t = L/V$, where L is the length of the sewer and V is the full-pipe flow velocity computed by using the Darcy-Weisbach formula (Eq. 45) with f given by the Colebrook-White formula

$$\frac{1}{\sqrt{f}} = -\log_{10} \left[\frac{k}{14.8R} + \frac{0.89v}{gR\sqrt{RS}} \right] \quad (92)$$

Alternatively, Manning's formula can also be used to compute the velocity.

The inflow hydrograph of a sewer is the combination of the outflow hydrographs, with appropriate time lag as computed by the time of travel, of the inlets and sewers joining at the upstream end of the sewer being considered. This inflow hydrograph is then routed through the sewer by using the continuity equation

$$\frac{I_1 + I_2}{2} - \frac{Q_1 + Q_2}{2} = \frac{s_2 - s_1}{\Delta t} \quad (93)$$

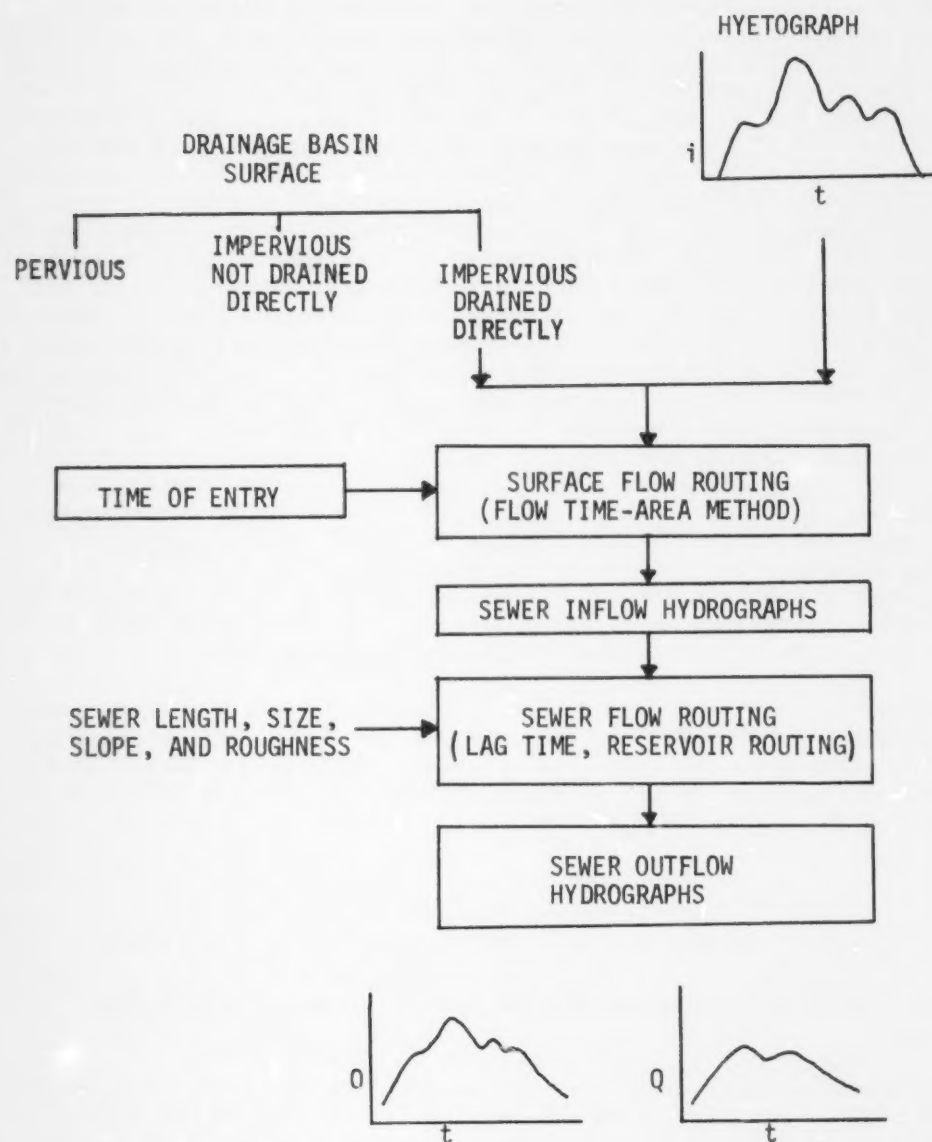


Fig. 33. Schematic diagram of TRRL method

in which I is the inflow rate as given by the inflow hydrograph, Q is the outflow rate of the outflow hydrograph, s is the storage in the sewer, and the subscripts 1 and 2 refer to the beginning and end of the time interval Δt , respectively. A storage-discharge relationship is then supplemented to Eq. 93 to give the outflow rate. Originally, Watkins suggested to use the recession part of recorded runoff hydrograph to establish the storage-discharge relationship. In a later version, it was suggested to approximate this relationship using the Darcy-Weisbach formula (Eq. 45) with f given by Eq. 92 assuming instantaneously the sewer flow is steady and uniform with a slope equal to the sewer slope. A linear interpolation between the values of hydraulic radius and flow area was suggested to avoid time consuming iterative solution. A block diagram of the structure of the TRRL method is shown in Fig. 33.

Obviously, the TRRL method would be most successful for drainage basins of nearly equal in sizes of impervious and pervious areas and for rainstorms of moderate intensities and durations. To improve the method to account for the contributions of pervious areas on runoff, recently in the United States Terstriep and Stall (1974) proposed to estimate the runoff from grassed area in a manner similar to the paved area, and in Great Britain to approximate the runoff from pervious area by a linear reservoir model. However, these recent modifications of the TRRL method are not included in the comparative study in Section X because of inadequate information at the time of writing. From the theoretical viewpoint, the criticisms on the TRRL method are numerous (Heeps and Mein, 1973). For instance, in considering the sewer flow, the velocity is composed using Eqs. 47 and 92 assuming a full-pipe flow, whereas in computing the sewer storage, the sewer is assumed to have unlimited storage capacity as required by Eq. 93. In general, the TRRL method does not take into account the actual physics of storm water flow on the land surfaces and in sewers. The method should be considered as empirical rather than theoretical.

IX-5. CINCINNATI URBAN RUNOFF MODEL

The University of Cincinnati Urban Runoff (UCUR) Model is basically a hydrograph routing model developed under a grant from the U.S. Environmental Protection Agency (University of Cincinnati, 1970; Papadakis and Preul, 1972). The model has many similarities to the Chicago method and hydraulically it is essentially a linear kinematic-wave model. As in the Chicago method, infiltration is subtracted from rainfall using Horton's formula (Eq. 34) with the aid of Jens' (1948) curves, and the depression storage by using the same exponential function (Eq. 35) as in the Chicago method. No infiltration is allowed from water stored on the land surface. Interception and evapotranspiration are neglected. The resulting hyetograph of rainfall excess can then be used for routing. The basin is subdivided into a number of subcatchments each having homogeneous infiltration characteristics. At any instant the rainfall is assumed uniformly distributed over the entire basin but the abstractions are different for different subcatchments.

In routing the storm water, the overland flow is computed by using Manning's formula (Eq. 46) coupled with an empirical detention storage function. Assuming uniform flow with depth equal to the hydraulic radius, Manning's formula combined with the continuity equation gives a relationship for detention storage per unit width of the overland strip under equilibrium condition, d_e , as

$$d_e = C_e \frac{(i_e n L)^{0.6} L}{S^{0.3}} \quad (94)$$

in which i_e is the overland flow supply rate which is equal to the intensity of rainfall excess; n is the Manning roughness factor; L and S are the length and slope of the overland strip, respectively; and C_e is a dimensional coefficient, equal to $0.625 \text{ sec/m}^{0.3}$ in SI system with length in m and time in sec, and equal to $0.437 \text{ sec/ft}^{0.3}$ in English system or 0.00073 if i_e is in in./hr. The empirical detention storage function to be used together with Eq. 46 to solve for the overland flow hydrograph is

$$h = d + 0.6d \left(\frac{d}{d_e}\right)^{0.3} \quad (95)$$

in which d is the detention storage and h is the flow depth at the exit of the overland strip. During recession the ratio d/d_e is assumed to be unity. It should be noted here that in the UCUR method the overland flow depth does not include the depth of water for depression storage.

The gutter outflow is assumed equal to the sum of the upstream inflow, Q_u , and lateral inflow from overland at the same time increment; i.e.,

$$Q = Q_u + q_o L \quad (96)$$

in which q_o is the overland flow supply per unit length of gutter and L is the gutter length. No time lag of gutter flow is considered. Thus, despite the required detailed data for overland surfaces (length, slope, and Manning's n of the overland surfaces), the required gutter data is rather simple: only the length of the gutter. Neither the slope nor the cross section of the gutter is needed. In Eq. 96 it is implicitly assumed that the overland surface connected to the gutter is approximately rectangular in shape. Because the effects of the slope, shape, and surface roughness of the gutter are not accounted for, it can be expected that the UCUR method is not reliable when the gutter storage is important, such as the cases with small gutter slope and street cross slope and with heavy rainstorms.

For sewer flows, no pipe storage is considered. The hydrographs are simply lagged, without changing shape, by an average flow velocity, V_{av} ,

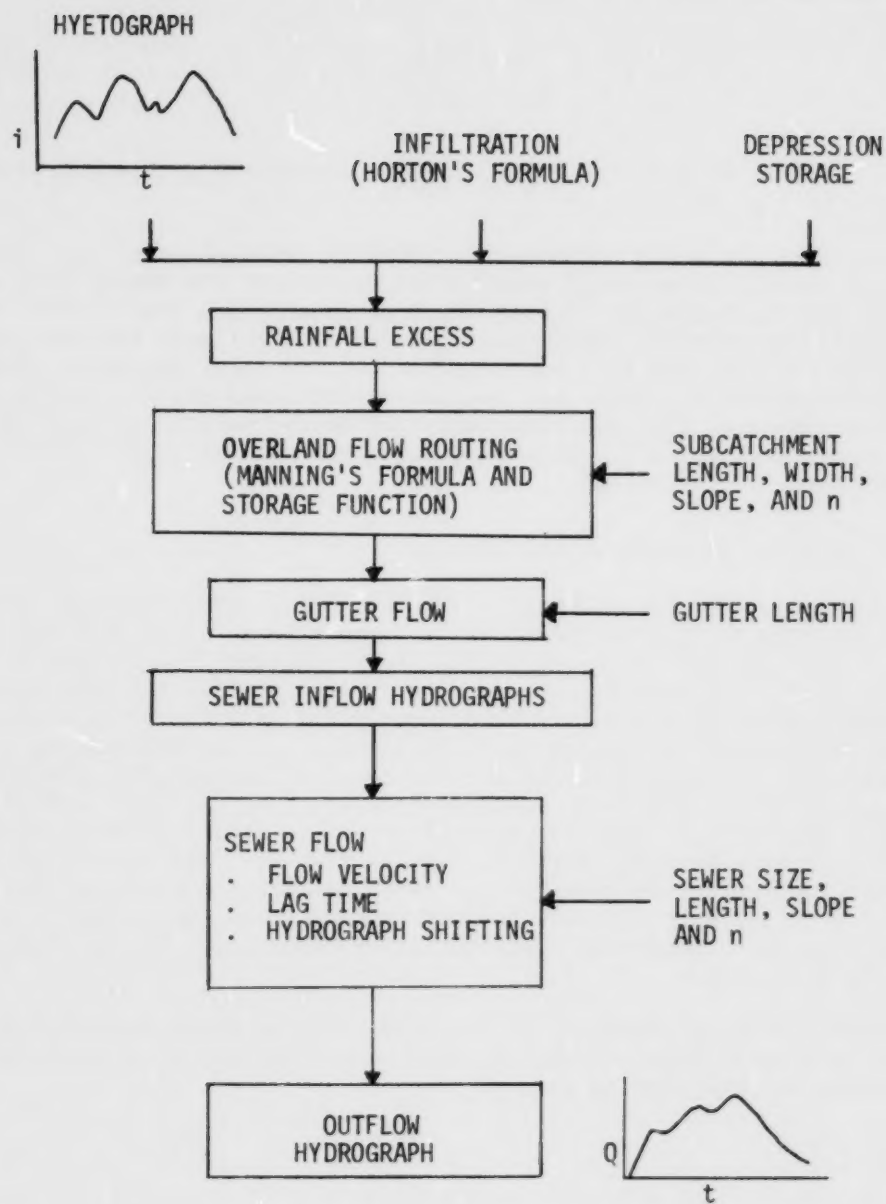


Fig. 34. Schematic diagram of Cincinnati urban runoff model

which is the weighed velocity of the flow velocity of the sewers computed by using Manning's formula; i.e.,

$$V_{av} = \frac{\sum V_j Q_j}{\sum Q_j} \quad (97)$$

A block diagram showing the major elements considered in the UCUR model is given in Fig. 34.

The procedure has been programmed for digital computer solutions. The method is rather tedious and complicated in view of the assumptions made to simplify the hydraulic routing of storm runoffs and the accuracy that the method can provide. Heeps and Mein (1973, 1974) modified the program to reduce the computer time and to allow defined sewer networks. They used Newton-Raphson's iteration instead of trial-and-error to solve the overland and sewer flow equations and omitted the weighed average velocity computation (Eq. 97). Their modifications substantially improve the applicability of the method.

IX-6. DORSCH HYDROGRAPH-VOLUME-METHOD

The Dorsch Hydrograph-Volume-Method (Klym et al., 1972; Vogel and Klym, 1973) is a flow prediction model and in most aspects it is more sophisticated than any one of the preceding five models briefly described in this section. Its data requirement is as extensive as the UCUR model and considerably more than the other four previously described methods (Table 6). The Dorsch method takes hyetographs as its rainfall input and a statistical method can be adopted for determination of frequent urban rainstorms. Different subcatchments can have different hyetographs. Interception and evapotranspiration are neglected and infiltration is computed by using Horton's formula (Eq. 34) with water from rainfall and surface storage, and allowing different parameters for different subcatchments. Like the UCUR method, each subcatchment necessarily has homogeneous characteristics within it. Gutters and inlets are ignored.

The surface flow is routed as in the Chicago hydrograph method using Manning's formula together with a continuity equation in storage form to account for the surface storage,

$$\frac{\partial h}{\partial t} = i - f - \frac{Q}{A_s} \quad (98)$$

in which i is the rainfall intensity; f is the infiltration rate; A_s is the surface area and Q is the discharge from the surface. Before applying Eq. 98 to evaluate the subcatchment runoff, the amount of depression should have already been accounted for. Equation 98 implies that the depth of water changes at the same rate over the entire surface area.

No consideration is given on changes of Manning's n for shallow depth and due to rainfall. The runoffs, Q , from the subcatchments become the inflow into the sewers.

Sewer flows are routed by using the diffusion approximation of the St. Venant equations (Sevuk, 1973; Yen, 1973a), i.e., neglecting the inertia terms (first two terms) in Eq. 43. By further ignoring the effects of the lateral flow q_0 and for small slope with $\cos \theta \approx 1$, the momentum equation can be simplified from Eq. 43 in finite difference form as

$$\frac{\Delta h}{\Delta x} = S_o - S_f \quad (99)$$

where S_f is estimated by

$$\frac{S_f}{S_o} = \left(\frac{Q}{Q_o}\right)^2 \quad (100)$$

in which Q_o is the steady uniform normal discharge corresponding to S_o , and presumably calculated by using Manning's formula or Darcy-Weisbach's formula with the resistance coefficient from the Colebrook-White formula (Vogel and Klym, 1973). The corresponding continuity equation in finite difference form can be obtained from Eq. 52 using the x - t plane shown in Fig. 23,

$$(A_D - A_A + A_C - A_B) \frac{1}{2\Delta t} + (Q_B - Q_A + Q_C - Q_D) \frac{1}{2\Delta x} = q_L \quad (101)$$

The flow equations for the sewer junctions used in the Dorsch method are

$$\Sigma[Q]_{\text{inflow}} = \Sigma[Q]_{\text{outflow}} \quad (102)$$

$$\Sigma\left[\left(z+h + \frac{v^2}{2g}\right)Q\right]_{\text{inflow}} \geq \Sigma\left[\left(z+h + \frac{v^2}{2g}\right)Q\right]_{\text{outflow}} \quad (103)$$

$$\min(z+h)_{\text{inflow}} \geq \max(z+h)_{\text{outflow}} \quad (104)$$

In other words, even for large junctions the storage capacity of the junction is ignored, and the junction loss is not directly and explicitly accounted for.

As in the case of solving the St. Venant equations, to solve Eqs. 99 and 101 for the sewer flow it is necessary to provide two boundary conditions in addition to the initial condition. For subcritical flow one is the

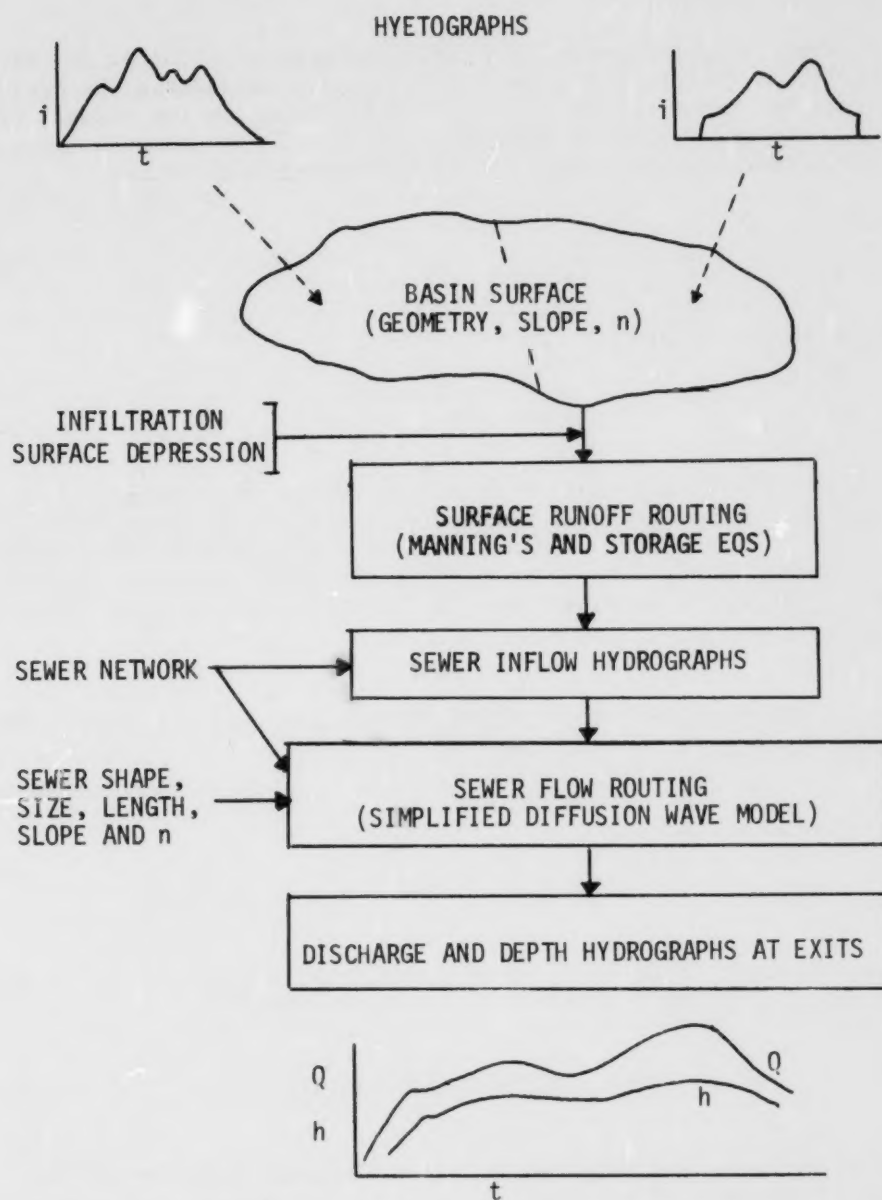


Fig. 35. Schematic diagram of structure of Dorsch hydrograph-volume-method

upstream boundary condition and the other is the downstream boundary condition which actually is unknown and part of the solutions being sought. Thus, a costly simultaneous solution of the flow equations for all the sewers and junctions is necessary as discussed in Sec. VII-2. In the Dorsch method this costly simultaneous solution technique is avoided by the assumption of Eq. 100, which prescribes the water surface profile in the sewer once the upstream condition is known, eliminating the need of a downstream boundary condition and permitting the solution to proceed in sequence towards downstream. However, this assumption also effectively eliminates the mechanism to realistically accounting for the downstream backwater effect. In other words, although the upstream backwater effect is accounted for, the computed water surface profile at the downstream end of the sewer is merely the result of the assumption instead of the true one. This is the reason why Dorsch method cannot account for reverse flow in sewers. Since the computed downstream flow condition is used in Eqs. 102 to 104 to determine the upstream condition of the following sewer, the error due to Eq. 100 can propagate and amplify as the computation proceeds. A block diagram of the major elements of the Dorsch method is shown in Fig. 35. Because the Dorsch method is a proprietary model, users should refer to the model developer for procedural details.

IX-7. EPA STORM WATER MANAGEMENT MODEL

The Storm Water Management Model (SWMM) was first developed jointly by Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc. (1971) under the sponsorship of the U.S. Environmental Protection Agency. Subsequent separate modifications and improvement of SWMM by the University of Florida researchers and Water Resources Engineers resulted in the EPA SWMM version and the WRE SWMM version, respectively. The version compared in this report is the nonproprietary EPA SWMM (Heaney et al., 1973; Huber et al., 1975).

SWMM is a comprehensive urban storm water quantity and quality runoff prediction and management simulation model. The surface runoff is routed by using a linear kinematic wave approximation and the sewer routing is an improved nonlinear kinematic wave model incorporating some features of the nonlinear quasi-steady dynamic-wave model. Because SWMM is probably the best documented one among the recently developed models, details of the EPA SWMM can easily be found in reports by Metcalf & Eddy, Inc., et al. (1971), Heaney et al. (1973), and Huber et al. (1975).

In the six methods previously described in this section, for the first five at a given time only one hyetograph is permitted for a drainage basin. In other words, the rainfall is assumed uniformly distributed over the entire basin and no areal variation is permitted. In SWMM as well as in the Dorsch method several hyetographs applied to different subcatchments at any given instant can be specified. Within each subcatchment, the rainfall and area physical conditions are assumed homogeneous. Like other methods, interception and evapotranspiration are neglected.

Routing of overland flow in subcatchments is the same as the Dorsch method described in Sec. IX-6. Infiltration is accounted for by using Horton's formula (Eq. 34). Different degrees of permeability and different infiltration parameters for Horton's formula may be applied to different subcatchments. Infiltration is allowed for water stored on the surface in addition to rainfall. Overland flow is assumed not to occur until depression storage is filled. However, in SWMM, if not specified, by default one-quarter of the impervious surface area is assumed to be of zero depression storage. Overland flow is assumed to be one-dimensional with its depth being constant along its length at any given instant. A quasi-steady flow routing is applied to the overland flow using the continuity equation (Eq. 98) and Manning's formula (Eq. 46) with the depth (excluding the depth for depression storage) equal to hydraulic radius. Within each time interval the flow is assumed steady and uniform. The subcatchment data required include the length, width, slope, surface roughness, and infiltration parameters and percent imperviousness.

Unlike the Dorsch method which performs no gutter routing and requires no data on gutters and inlets, in SWMM the gutter flow is routed using the quasi-steady storage routing approach with Manning's formula (Eq. 46) and the storage continuity equation

$$(h_{t+\Delta t} - h_t) / \Delta t = (Q_{in} - Q_{out}) / \frac{1}{2} (A_{s_{t+\Delta t}} + A_{s_t}) \quad (105)$$

Each gutter is assumed to be fed uniformly along its length by the lateral flow from overland surface. No direct rainfall, infiltration or depression storage is considered for gutters. The gutter outflows into an inlet are combined to give the inlet hydrograph for sewer routing. No consideration is given on the effect of inlets in transforming the gutter outflows into the sewer inflow hydrograph, and no data on inlet is required.

The sewer routing is by a modified nonlinear kinematic-wave scheme. The continuity equation (Eq. 52) and Manning's formula (Eq. 46) are used with the slope assumed equal to friction slope, and the flow is assumed to be steady within each time interval. No lateral flow is allowed within the sewer length and any subsurface flows to a sewer is assumed to flow into the upstream junction and combined with the junction infiltration water. The sewer can be of any cross sectional shape. The continuity equation is put into a finite difference form, using the x-t plane shown in Fig. 23 with $\Delta x = L$ = sewer length, as follows

$$\frac{(1-W_t)(A_D - A_A) + W_t(A_C - A_B)}{\Delta t} + \frac{(1-W_x)(Q_B - Q_A) + W_x(Q_C - Q_D)}{L} = 0 \quad (106)$$

in which Q is the discharge; and A is the flow cross sectional area. The time derivative is weighted W_t at the downstream station and the

spatial derivative is weighted W_x at the end of Δt . Subsequently Eq. 106 is normalized by the full conduit flow area and discharge A_f and Q_f , respectively. By assuming steady uniform condition and using Manning's formula (Eq. 46) a single curve representing the relationship between Q/Q_f and A/A_f for the conduit can be established. With this nondimensional discharge-area curve replacing Manning's formula and the normalized (only for A and Q) form of Eq. 106, numerical solutions are then obtained with known initial and upstream boundary conditions using a four-point implicit finite difference scheme.

Since no downstream boundary condition is required for the solution, SWMM cannot account for the downstream backwater effect when the sewer flow is subcritical. Nevertheless, to improve the solution accuracy, a genius approximation is introduced. Although the $Q/Q_f - A/A_f$ curve is established assuming steady uniform flow, in seeking the solution the value of Q_f is actually computed by using Manning's formula with the friction slope S_f estimated by using a quasi-steady dynamic-wave approximation (Yen, 1973a) of the St. Venant equation, i.e., dropping the local acceleration term (the first term) in Eq. 43. Thus, using the $x-t$ plane shown in Fig. 23, the friction slope for Q_f is

$$S_f = S_o - \frac{h_A - h_D}{L} + \frac{v_A^2 - v_D^2}{2gL} \quad (107)$$

The use of Eq. 107 to estimate Q_f partially and approximately accounts for the downstream backwater effect, though always one time step late. Nonetheless, since the downstream boundary condition is not truly accounted for, it is recommended in SWMM that for a sewer with a large downstream storage element for which the backwater effect is severe, the water surface be assumed as horizontal from the storage element going backward until it intercepts the sewer invert. Moreover, when the sewer slope is steep, presumably implying high velocity supercritical flow, the flood may simply be translated through the sewer without routing. Also, if the backwater effect is expected to be small and the sewer is circular in cross section, the gutter flow routing method may be applied to the sewer as an approximation. No hydraulic jump or drop is considered within a sewer.

In SWMM large junctions with significant storage capacity and storage facilities are called storage elements, equivalent to the reservoir-type junctions in the ISS model. Only the continuity equation is used in storage element routing. No dynamic equation is considered except for the cases with weir or orifice outlets. Small junctions are treated like the point-type junctions in the ISS model.

A diagram showing schematically the structure of the water quantity part of SWMM (roughly corresponding to Runoff Block and Transport Block) is given in Fig. 36.

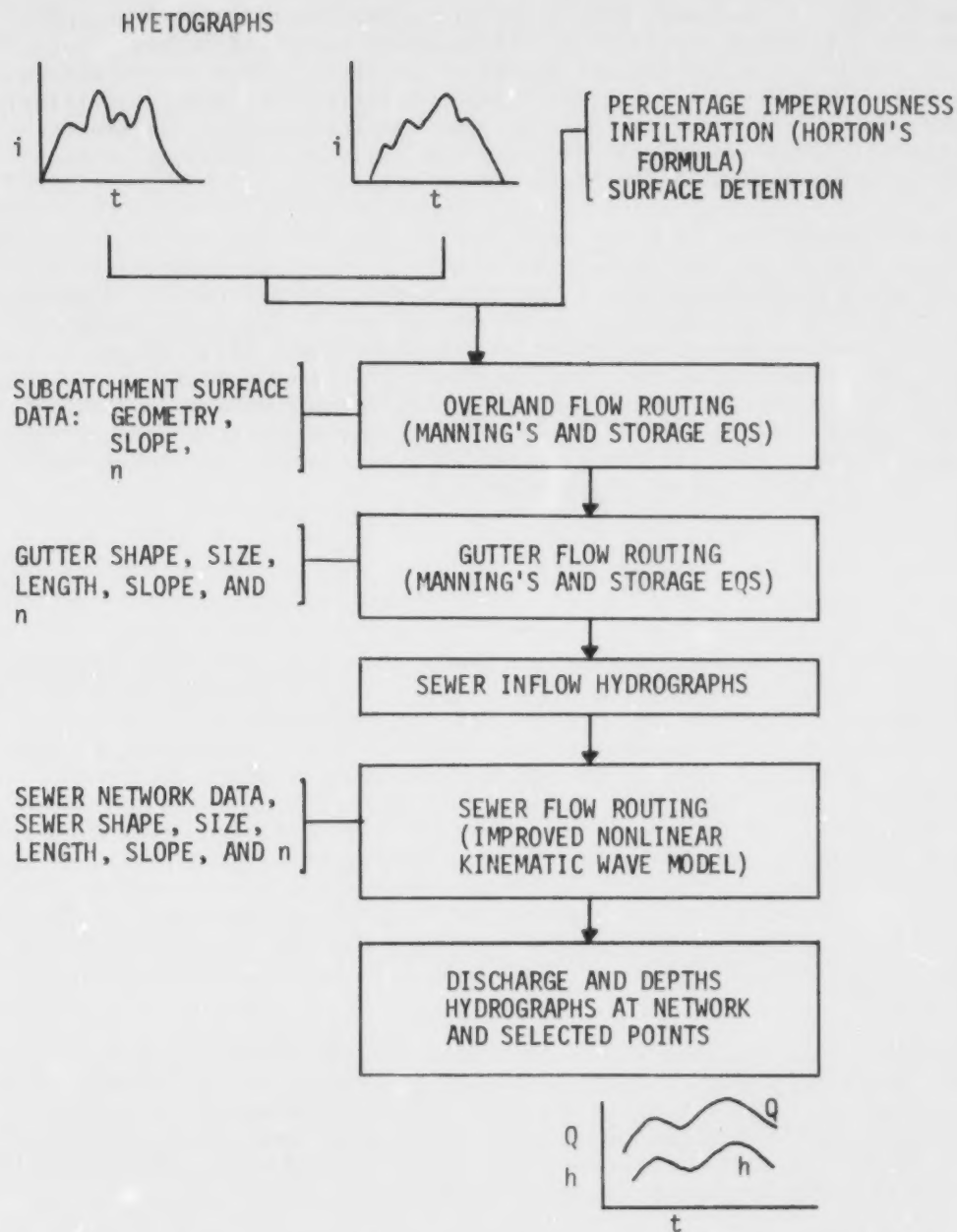


Fig. 36. Schematic diagram of water quantity routing of SWMM

IX-8. ILLINOIS URBAN STORM RUNOFF METHOD

The Illinois Urban Storm Runoff (IUSR) method consists of three components: the Illinois Surface Runoff Model as described in Section VI and App. B, the Illinois Storm Sewer System Simulation Model as described in Section VII and a user's manual (Sevuk et al., 1973), and the Illinois Sewer System Water Quality Model as described in Section VII and App. C. A block diagram showing the structure of the IUSR method has been given in Fig. 20. Any one of the three component models can be used independently to interface with other runoff models provided the data requirement is compatible.

For the runoff quantity evaluation, only the Illinois Surface Runoff Model and the ISS Model of the IUSR method are involved. The rainfall is first subtracted by infiltration and then routed through overland surfaces, gutters, inlets, sewers and storage junctions. Within the drainage basin areal variations of rainfall, infiltration, surface detention and surface roughness are allowed. From the hydraulics viewpoint, both the surface and sewer routings of the IUSR method are of higher order than any one of the seven methods previously described in this section. In fact, with the possible exception of an unpublished French model (SOGREAH, a proprietary model for which details cannot be obtained for evaluation) the IUSR method is the only known model that actually accounts for both the downstream and upstream backwater effects of the sewers and automatically computes reversal flows, when they occur, without making additional unrealistic assumptions. The IUSR method permits more freedom than other methods in representing more realistically the characteristics of rainfall, drainage basin, and sewer system. However, its data and computer time requirements are also most extensive among the eight methods evaluated in this study. It is the only one that requires data on inlets since it is also the only one that considers the hydraulic effect of inlets on storm runoff.

In a first glance, it appears that the IUSR method could be applied only to tree-type sewer systems with only three conduits joining at a junction, and with flow regulators allowed only at the sewer system outlet. Actually, the method can be applied to sewer network with as many sewers joining at a junction, so long as the sewers in excess to three can be treated as direct inflow into or outflow from the junction, and to sewer loops so long as the flows at the ends of one of the sewers in the loop can be treated as direct inflow or outflow (with prescribed or approximate functions) into or out from the respective junctions. Allowing regulators only at the sewer system outlet in the ISS model is actually an advantage rather than disadvantage in obtaining the solution. In the previously described seven methods, because the downstream backwater effect is not considered, solution for sewer flow follows a cascading model solving for the sewers one by one, proceeding towards downstream. For the ISS model, as explained in Sec. VII-2, because the downstream backwater effect is accounted for, the sewers within a system are solved as a unit. The flow regulators with their known hydraulic expressions facilitate a means to decompose a large system into smaller subsystems

for easier handling in seeking the solution. Since the ISS model computer program permits continuous repeated loading of sewer systems, the solution for a large system is achieved through a cascading model of the subsystems of the decomposed system in sequence instead of the individual sewers in sequence as for the other seven methods.

Similar to the other methods, the IUSR method handles surcharges in sewer systems with assumptions. In the ISS model, when surcharge occurs, the model still treats the flow in the sewer as an open-channel flow by tentatively using a larger commercial pipe size to replace the actual pipe for the computation. At present, the computer program can handle only circular conduits. For non-circular sewers, the user should use their equivalent hydraulic radius. All the three component models of the method permit repeated continuous loading, therefore the models can be used in real-time simulation, although such use has not been tested.

SECTION X

EVALUATION OF MODELS

The eight urban storm runoff simulation models described in Section IX are compared using four rainstorms on the Chicago Oakdale Avenue Drainage Basin. Based on the recorded hyetographs the runoff hydrographs are predicted by using the eight simulation models (the rational method gives only the peak discharge) and the results are compared with the measured hydrographs. Such an evaluation using only a few measured rainstorms of course is limited in scope and one should be cautioned on the generalization of the conclusions.

Theoretically, it would be desirable also to compare the different models on drainage basins having their sizes order-of-magnitude bigger than that of the Oakdale Basin. However, in view of the requirements of data details and accuracy for the more sophisticated models and the computer time required for large basins, comparison using many rainstorms on large basins would be extremely costly and time consuming. For applications some errors on the data and results may be acceptable, whereas for the purpose of model evaluation and comparison excessive errors are intolerable. Furthermore, from the hydrodynamic viewpoint it is more desirable to evaluate the models with smaller basin size with sufficient and accurate details than to larger basins, because for the larger basins the local effects tend to be averaged out and not as clearly reflected in the basin outflow hydrographs as for the smaller basins. For the Oakdale Avenue Basin, because of its small size and proximity, the writers were able to check in detail the field situation needed for the more complex models within the time and budget allowance. With the presently available data, comparison using a larger basin would not provide additional new conclusions. In fact, as will be discussed later, the testing on the Oakdale Basin suggests that a testing of the surface runoff part of the models on a smaller area with accurate data is highly desirable.

X-1. HYDROGRAPHS FOR THE EIGHT METHODS

The four rainstorms together with their respective measured runoff hydrographs for the Oakdale Avenue Drainage Basin in Chicago selected for the comparison study of the eight methods are the rainstorms of May 19, 1959, July 2, 1960, April 29, 1963, and July 7, 1964. These four rainstorms are chosen based on the following reasons: (a) The measurements of rainfall and runoff are supposedly reliable as recommended by Tucker (1968). (b) The rainfall is relatively heavy and hence presumably the measurements are relatively accurate. (c) The rainstorms have been used by previous investigators for establishment of their methods or for testing. The measured hyetographs and hydrographs are taken from a report by Tucker (1968) and reproduced in Figs. 37 to 40. The physiographic characteristics of the Oakdale Drainage Basin have been described in Section V and supplementary information can be found elsewhere (e.g., Tucker 1968).

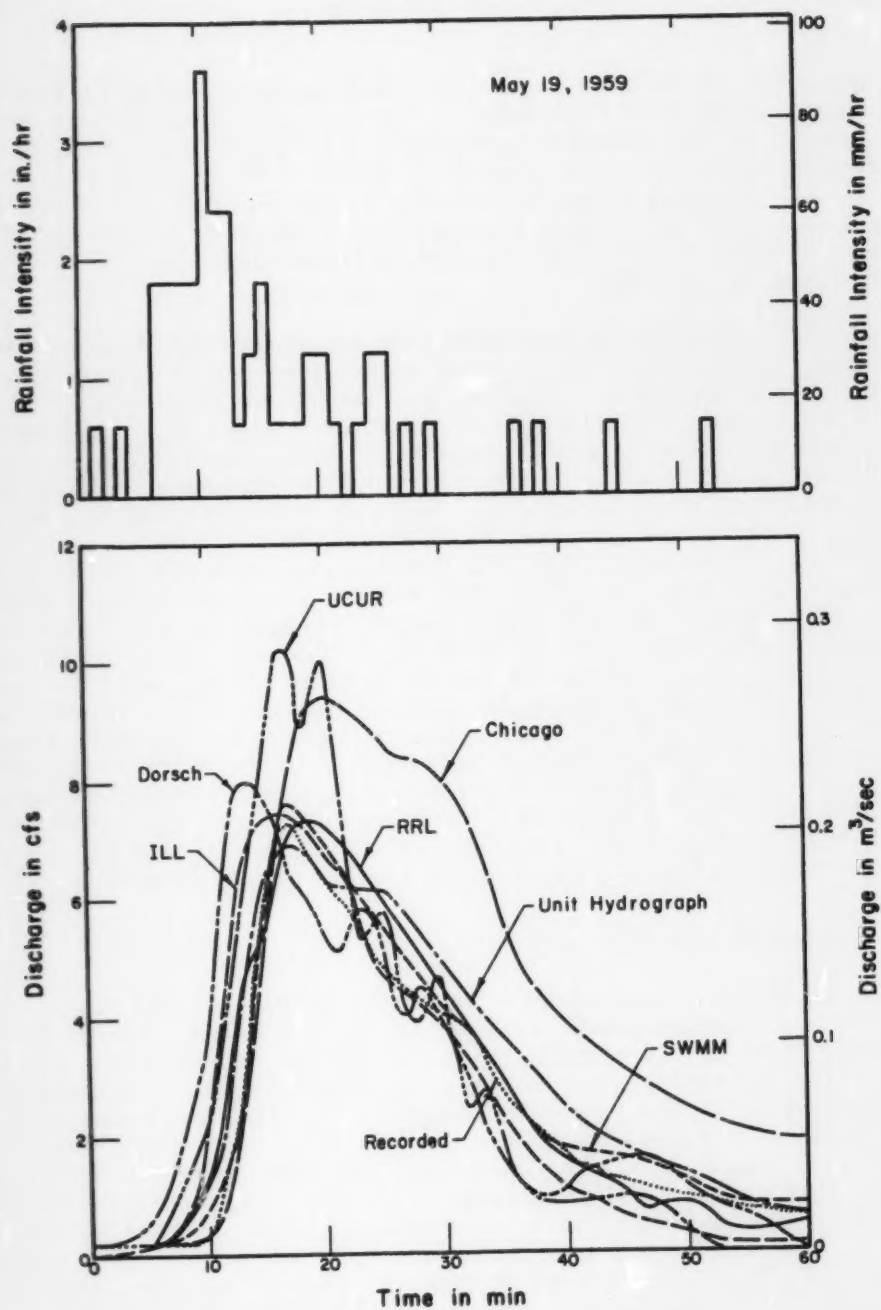


Fig. 37. Hyetograph and hydrographs for May 19, 1959 rainstorm

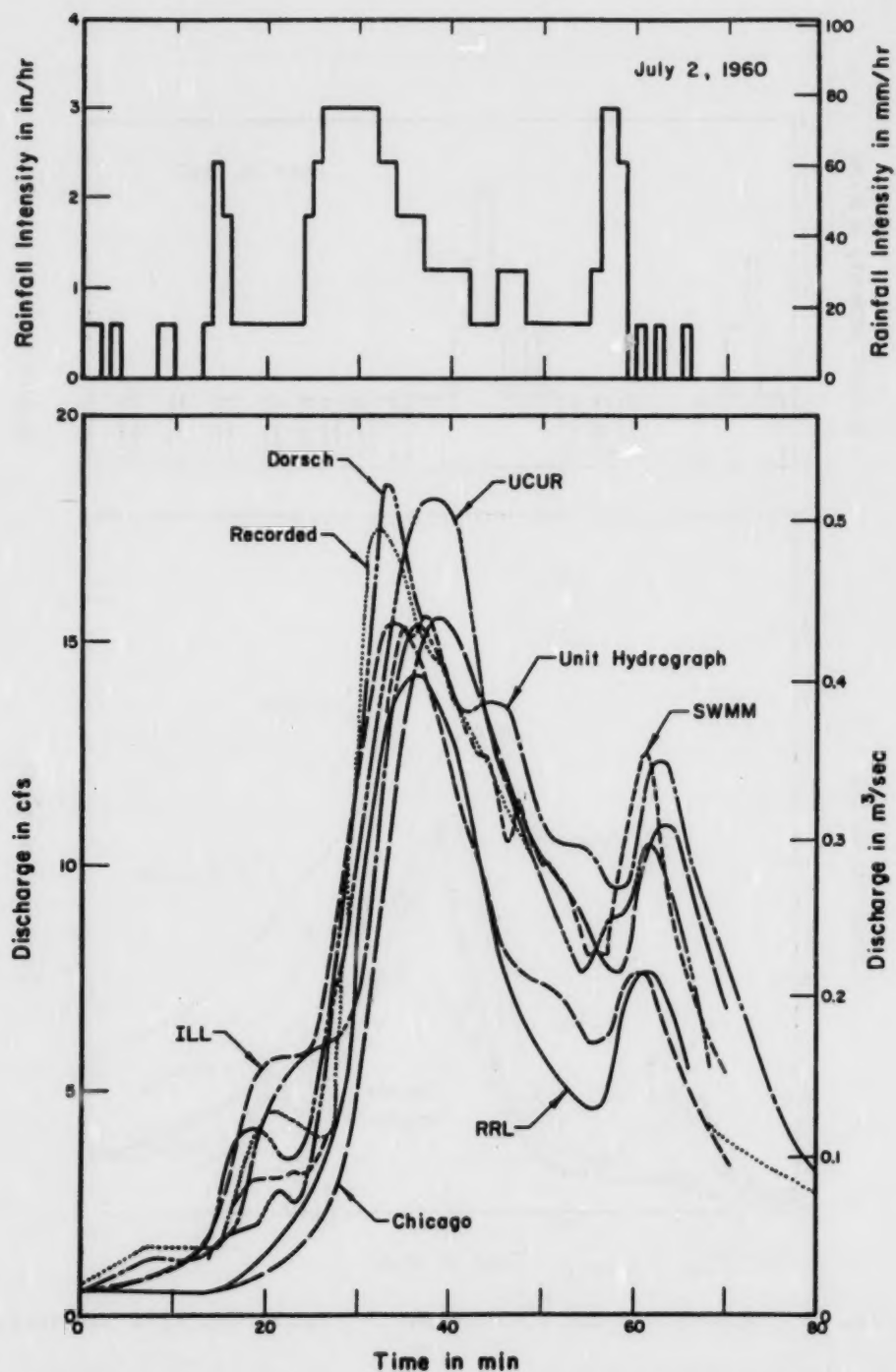


Fig. 38. Hyetograph and hydrographs for July 2, 1960 rainstorm

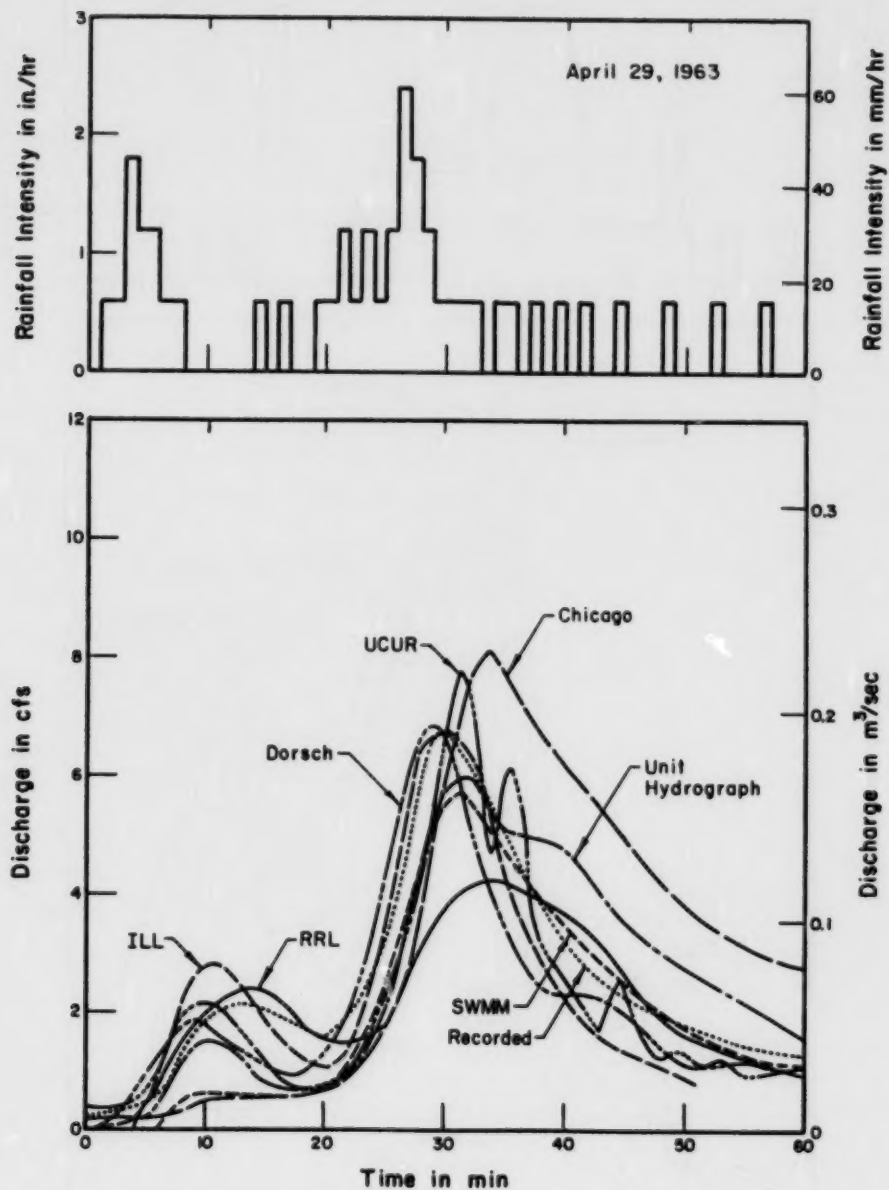


Fig. 39. Hyetograph and hydrographs for April 29, 1963 rainstorm

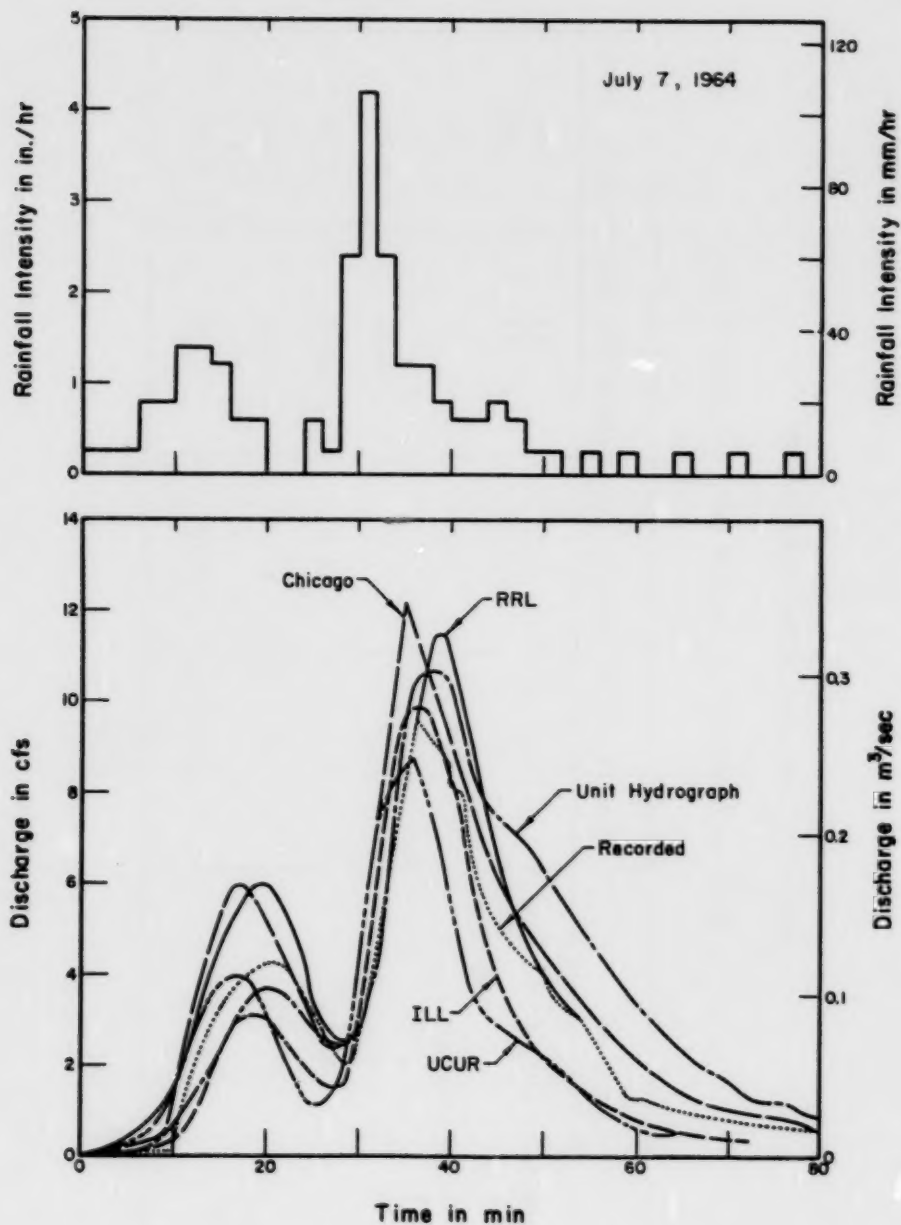


Fig. 40. Hyetograph and hydrographs for July 7, 1964 rainstorm

Table 7. RATIONAL METHOD COMPUTATION

Area = 12.9 ac or 0.052 km², Runoff Coefficient C = 0.60

Rainstorm	May 19, 1959	July 2, 1960	April 29, 1963	July 7, 1964
Period in min for i, from to				
Duration, min	0 20	15 35	13 33	18 38
i	20	20	20	20
in./hr	1.17	1.77	0.75	1.29
mm/hr	29.7	44.9	19.0	32.8
cfs	9.1	13.7	5.8	10.0
Q _p m ³ /sec	0.26	0.39	0.16	0.28

Table 8. COMPARISON OF URBAN STORM RUNOFF METHODS

Method	Rainstorm	May 19, 1959		July 2, 1960		April 29, 1963		July 7, 1964	
		Q_p cfs	t_p min	Q_p cfs	t_p min	Q_p cfs	t_p min	Q_p cfs	t_p min
Rational		9.1	0.26	13.7	0.39	5.8	0.16	10.0	0.28
Unit hydrograph		6.9	0.20	15.3	0.43	6.0	0.17	10.7	0.30
Chicago		9.3	0.26	15.5	0.44	8.2	0.23	12.2	0.35
TRRL		7.3	0.21	14.2	0.40	4.4	0.12	11.5	0.33
UCUR		10.2	0.29	18.2	0.52	7.8	0.22	8.8	0.25
DORSCH HVM		8.0	0.23	18.5	0.53	6.8	0.19	---	---
EPA SWMM		7.6	0.22	15.6	0.44	5.7	0.16	---	---
Illinois		7.4	0.21	15.4	0.44	6.7	0.19	9.9	0.28
Recorded		7.2	0.20	17.5	0.49	6.7	0.19	9.6	0.27
									36.5

(A) Rational method

The computation for the rational method is shown in Table 7. The time of concentration, t_c , for the 0.052 km² (12.9 ac) drainage basin is 23 min, determined using the monograph by Kerby (1959) for the flow on the subcatchment draining into the sewer Junction 117 (Fig. 16) plus the sewer flow time from Junction 117 to the basin outlet which is computed by using Manning's formula assuming barely filled gravity pipe flow. The actual time of concentration is probably shorter as indicated by the recorded hyetographs and hydrographs. Therefore, a time of concentration of 20 min is adopted here. The rainfall intensity i used in the rational formula is the average intensity of the recorded rainfall over a duration equal to t_c . The runoff coefficient C used is 0.60 which is the value one would adopt from standard tables (Chow, 1968, 1964; ASCE and WPCF, 1969) corresponding to the surface condition of the Oakdale Basin. As customarily done for the rational method, no abstraction is made from the rainfall and no adjustment of the C value is made to account for the preceding rainfall or antecedent surface wetting conditions. As shown in Table 8, the computed peak discharges by the rational method can be considerably different from the measured values.

(B) Unit hydrograph method

As discussed in the preceding chapter, only in rare cases that sufficient data are available for urban drainage basins to establish their respective unit hydrographs. Fortunately, the Oakdale Basin is one of those few that its unit hydrographs can be obtained. From the recorded data recommended by Tucker (1968), rainfalls of 10-min duration were selected together with the corresponding hydrographs to establish the 10-min unit hydrographs. The eight 10-min unit hydrographs so obtained are shown in Fig. 41. Based on these eight unit hydrographs the average 10-min unit hydrograph is plotted as shown in Fig. 41 and is used in this study for runoff prediction. The 1-min unit hydrograph is subsequently obtained by using the S-hydrograph method (Chow, 1964) as shown in Fig. 42 and checked against recorded data. The 2-min unit hydrograph can then be derived by adding one 1-min unit hydrograph to another 1-min unit hydrograph with a 1-min time lag. It should be mentioned here that in Fig. 41 the deviation among the eight hydrographs for the eight recorded 10-min rainfalls may reflect, in addition to data accuracy and linear approximation of a nonlinear physical phenomenon, the effect of changes in basin characteristics with season and time.

In applying the unit hydrographs of different durations to the four rainstorms to obtain the runoff hydrographs, the abstraction made from the total rainfall to give the rainfall excess is a rather subjective matter. As discussed in Sec. IX-2, in this investigation for the sake of consistency, Horton's formula (Eq. 34) is used to estimate the infiltration with the values of the parameters roughly the same as those used for the Illinois Surface Runoff Model. Also, the same base flow is added to both methods.

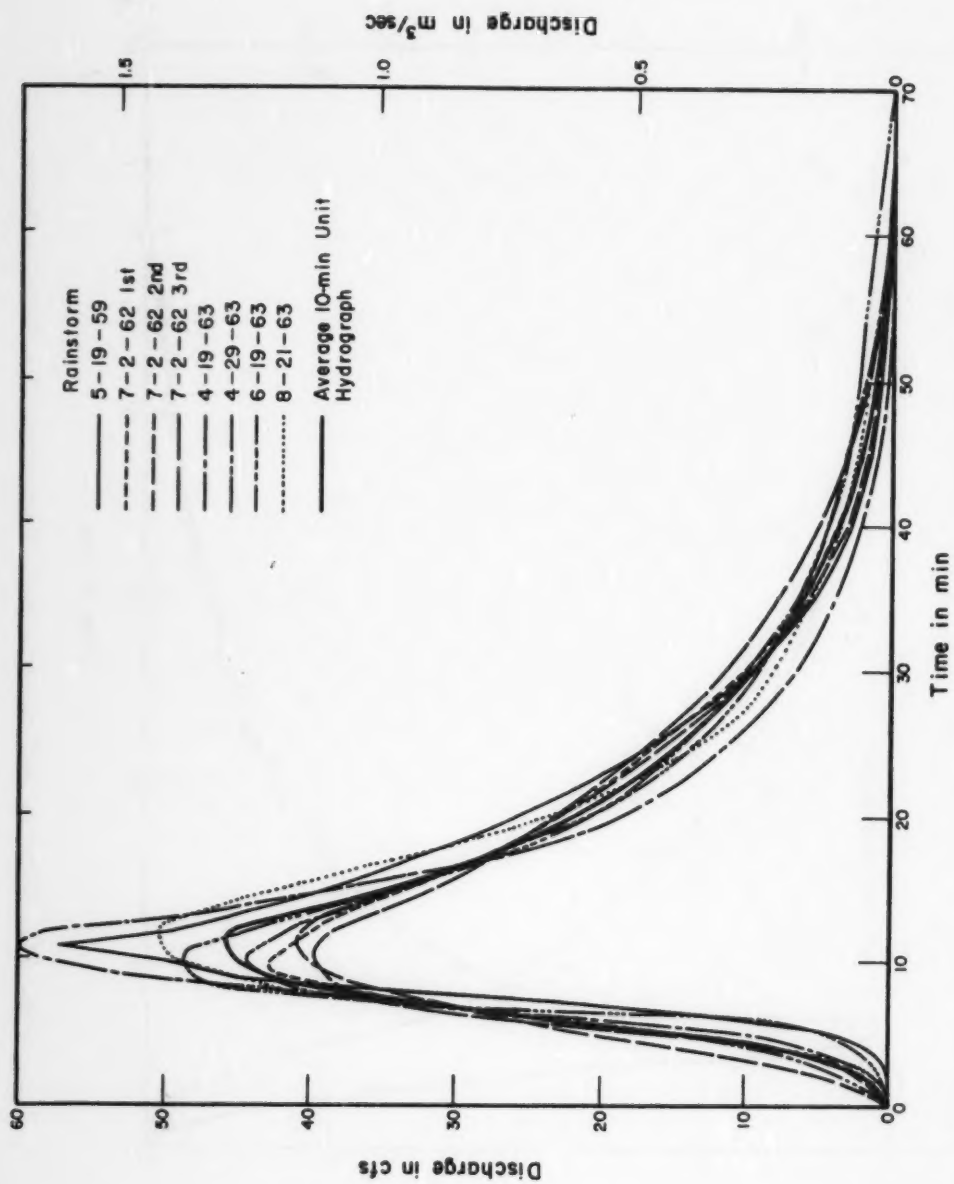


Fig. 41. Ten-min unit hydrograph for Oakdale basin

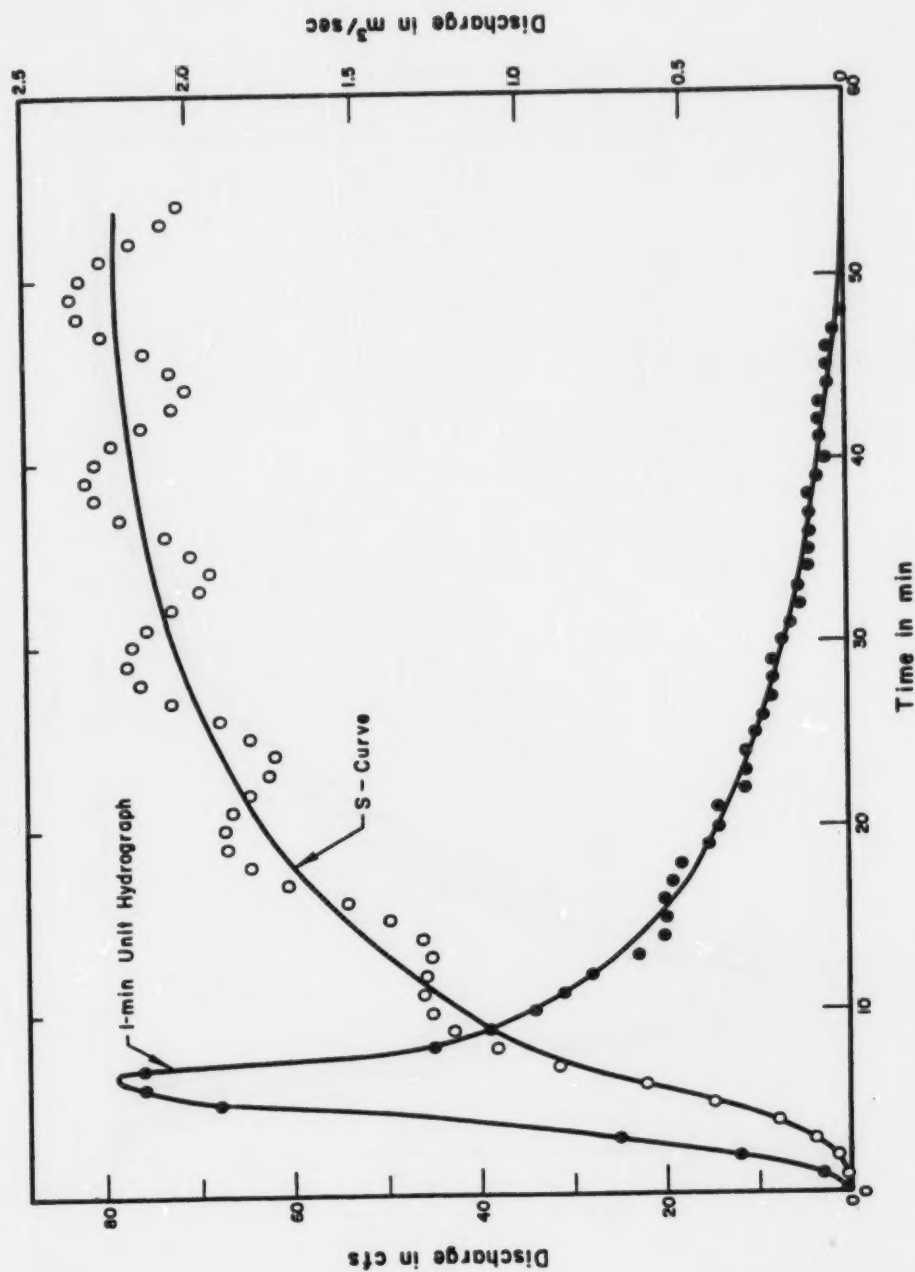


Fig. 42. S-curve and one-min unit hydrograph for Oakdale Basin

Table 9. HEADINGS FOR COMPUTATION OF UNIT HYDROGRAPH METHOD

Time	Recorded Rainfall	Infiltration		Rainfall Excess	Component Rainstorm		Runoff = Rainfall Excess x Unit Hydrograph Ordinates			Base Flow	Total Runoff
		Rate	Average Rate During Period		Duration	Excess	Component Rainstorm 1	Component Rainstorm 2	Component Rainstorm 3		

With the rainfall excess and base flow known, the standard procedure for using unit hydrographs is applied. The headings of the table used for the computations is shown in Table 9 and the results are shown in Figs. 37 to 40 respectively for the four rainstorms tested.

(C) Chicago hydrograph method

As mentioned in Sec. IX-3, the Chicago method actually has two versions. The time-offset version is not adopted in this study because the sewer layout of the Oakdale Basin is not favorable for using this version. Therefore, the storage routing version is used. The hydrograph for the July 7, 1964 rainstorm by the Chicago method shown in Fig. 40 was taken from the University of Cincinnati (1970) report. Because the computer program for Chicago method could not be released by the city of Chicago and only three other rainstorms were to be evaluated, it was not worthwhile in this study to write a computer program and hence the hydrographs for the other three rainstorms were computed by hand calculations. The computed results are plotted in Figs. 37 to 39 for comparison.

(D) Transport and Road Research Laboratory Method

The hydrographs by the British TRRL method for the four rainstorms tested have been computed by other investigators. The July 7, 1964 rainstorm was taken from Terstriep and Stall (1969) and plotted in Fig. 40. The July 2, 1960 rainstorm was taken from the University of Cincinnati (1970) report and shown in Fig. 38. For the other two, the May 19, 1959 and April 29, 1963 rainstorms, the hydrographs were taken from James F. MacLaren (1973) and replotted in Figs. 37 and 39. Of course the results would be different if the directly contributing impervious areas are taken differently from the values assumed by those investigators.

(E) University of Cincinnati Urban Runoff Model

The Hydrographs of rainstorms of July 2, 1960 and July 7, 1964 for the UCUR model were originally computed by the model developers (University of Cincinnati, 1970) and replotted here in Figs. 38 and 40. The hydrographs for the other two rainstorms, May 19, 1959 and April 23, 1963, were computed by James F. MacLaren (1973) for another comparative study and borrowed for this study as shown in Figs. 37 and 39.

(F) EPA Storm Water Management Model

The hydrograph of the July 2, 1960 rainstorm for SWMM shown in Fig. 38 was taken from the report by the model developers (Metcalf & Eddy, Inc., et al., 1971). It was calculated using the original version of SWMM. However, the quantity part of the original SWMM is essentially the same

as the later modified version called EPA SWMM. Hence this hydrograph can be used without recalculation. The hydrographs for the rainstorms of May 19, 1959 and April 29, 1963 were taken from the study by James F. MacLaren (1973) and replotted in Figs. 37 and 39.

(G) Dorsch Hydrograph Volume Method.

The Dorsch method is the only proprietary model compared in this study because the hydrographs for the May 19, 1959, July 2, 1960, and April 29, 1963 rainstorms are readily available (James F. MacLaren, 1973) and replotted in Figs. 37 to 39. For the same reason of proprietary no attempt was made to use the Dorsch HVM to calculate the hydrograph for the July 7, 1964 rainstorm.

(H) Illinois Urban Storm Runoff Method

The Illinois Urban Storm Runoff Method was used to compute the hydrographs for all the four rainstorms tested and the results are plotted in Figs. 37 to 40. Since details of this method are not available elsewhere they are described as follows as applied to the four rainstorms tested.

The measured rainfall recorded as the hyetograph is subtracted by infiltration to give the rainfall excess. The rainfall excess is then routed through the subcatchments using the nonlinear kinematic-wave approximation. When the rainfall rate recorded on the hyetograph is smaller than the infiltration capacity, the depth of water detained on the surface supplements the supply to infiltration.

As mentioned in Section V, the subcatchments of the Oakdale Basin consists of four types; roofs, lawns, sidewalks (including driveways), and street pavements. The asphalt street pavements are 4.27-m (14-ft) wide (one-half of street width) having an average cross slope of 2.7%. The infiltration through the street pavement is rather small and the values of f and k for Horton's formula (Eq. 34) used are 0.01 in./hr or 0.25 mm/hr and 10 hr^{-1} , respectively. Without adequate information on the initial wetting conditions the value of the initial infiltration f_0 for the four rainstorms is assumed to be 0.02 in./hr or 0.5 mm/hr. It was found that because of the small infiltration through the pavement, the results are not sensitive to the values of f_0 and k used.

Theoretically, the rainwater falling on the street pavements can be routed through the length of the pavement strips as described in Sec. VI-1. However, because of the short pavement length perpendicular to the gutter (4.3 m or 14 ft) the pavement flow time is less than half a minute and much less than the time intervals used for gutter flow computations. Consequently, such routing of flow through the street pavement would substantially increase the computer time and cost with essentially no improvement in the accuracy of the results. Furthermore, the kinematic wave routing method described in Sec. VI-1 does not account

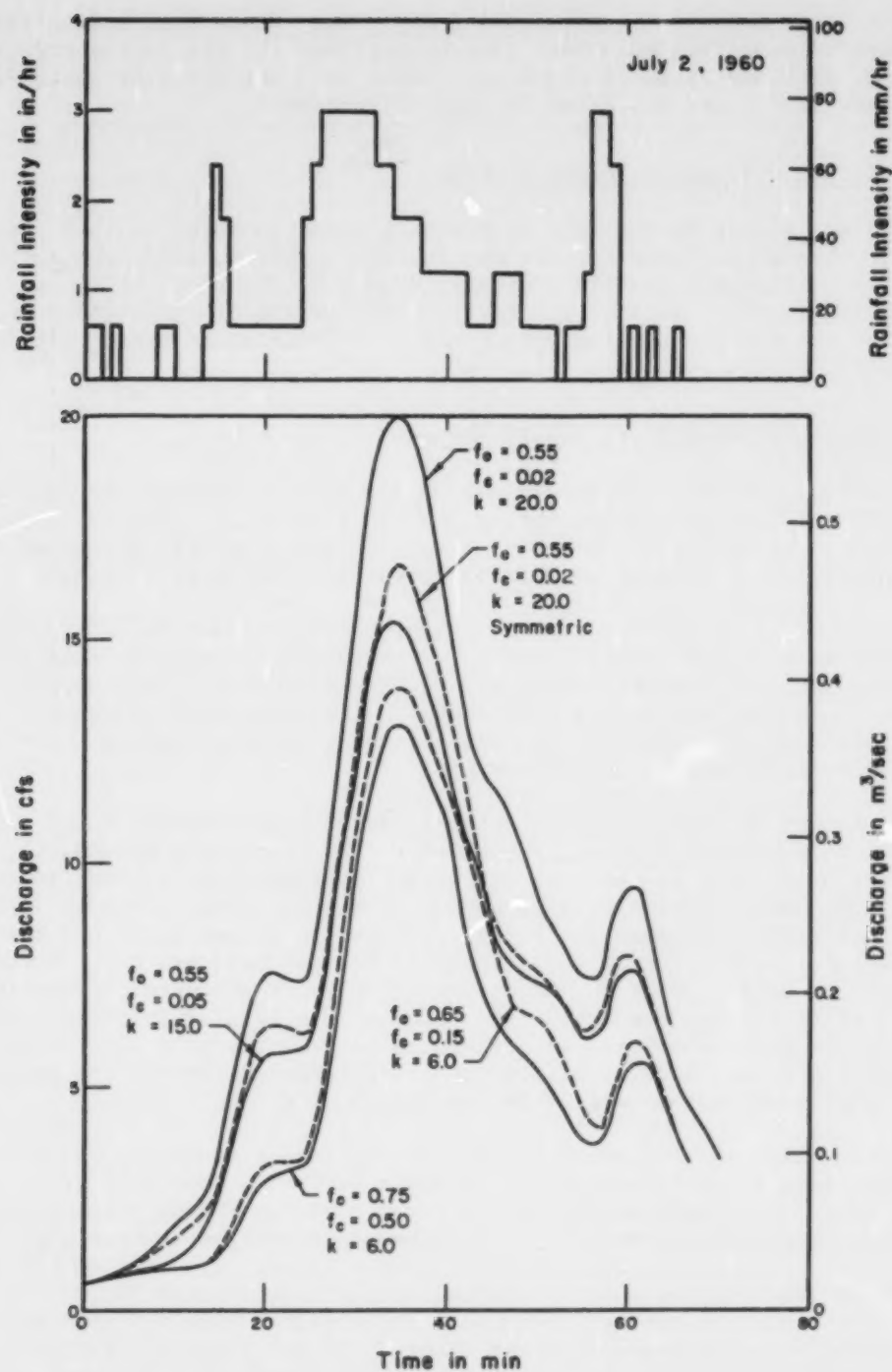


Fig. 43a. Sensitivity of computed hydrograph to Horton's infiltration parameters, Rainstorm of July 2, 1960

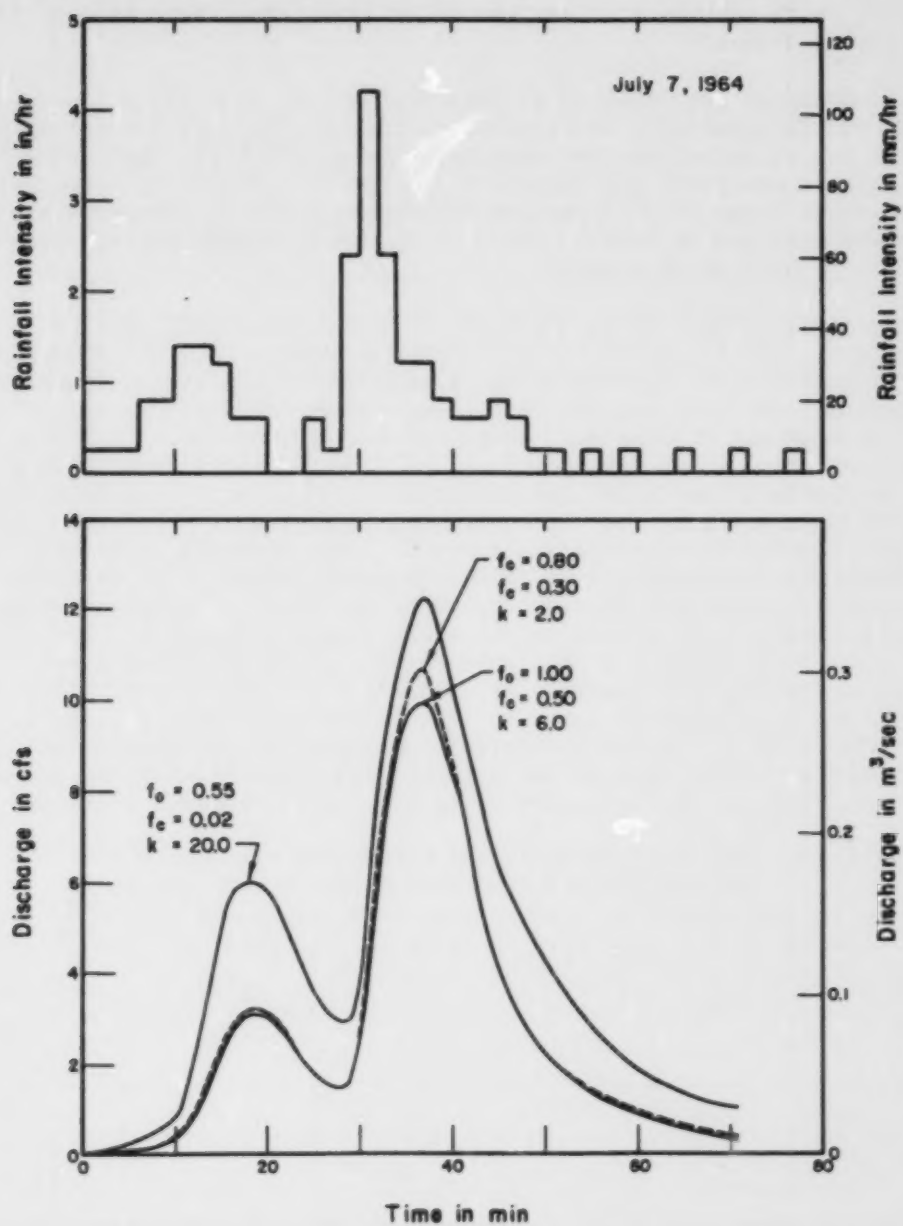


Fig. 43b. Sensitivity of computed hydrograph to Horton's infiltration parameters, Rainstorm of July 7, 1964

for the backwater effect from the gutter and hence the solution is only approximate. Therefore, it was decided not to perform the street pavement flow routing for the Oakdale Basin and the computed rainfall excess falling on the pavement is assumed to be transformed into lateral flow to the gutter directly.

As discussed in Sec. V-I, it is possible but impractical to consider in urban runoff prediction the details of distributions of the areas for roofs, lawns, and sidewalks separately and accurately. Therefore, in the present study for the Oakdale Basin for the purpose of flow routing these three types of surfaces are treated together as homogeneous overland surfaces and an average slope of 6% and hydraulic surface roughness of 0.01 ft or 3 mm is assumed.

As discussed in Sec. IV-2, theoretically, the final infiltration capacity f_c and the exponent k in Horton's formula (Eq. 34) should be nearly constant for the pavements as well as for the roofs, lawns and sidewalks. The only reasons that these values would change appreciably are the seasonal changes and changes of the drainage basin physical characteristics in the span of 5 years. However, sufficient data were not available to establish the values of f_c and k for the Oakdale Basin. Consequently a costly and time consuming trial-and-error process based on the recorded hydrographs was adopted. This approach is neither accurate nor foolproof and it is highly undesirable. This difficulty necessarily points out the importance of the need for detailed accurate data on land surface conditions in storm runoff prediction. For the four rainstorms tested, no suitable common values of f_c and k were found. In fact, the computed runoff hydrograph is sensitive to the values of f_c , k , and f_o assumed as shown in Fig. 43 for rainstorms of July 2, 1960 and July 7, 1964. Values of Horton's infiltration parameters used for the computed hydrographs of the Illinois Urban Storm Runoff Method shown in Figs. 37 to 40 are listed in Table 10. The variations of f_c and k

Table 10. VALUES OF INFILTRATION PARAMETERS USED FOR ILLINOIS URBAN STORM RUNOFF METHOD SHOWN IN FIGS. 37 TO 40

Rainstorm Parameter	May 19, 1959	July 2, 1960	April 29, 1963	July 7, 1964
f_o in./hr	1.00	0.55	0.80	1.00
f_o mm/hr	25	14	20	25
f_c in./hr	0.15	0.05	0.1	0.5
f_c mm/hr	3.8	1.3	2.5	12.7
k hr ⁻¹	10	15	15	6

estimated from recorded hydrographs for the four rainstorms tested due in addition to the error of using average values to represent the integrated effect over the entire basin of infiltration of individual component areas of different types, may also actually reflect the effects of movement of the rainstorms and nonuniform areal distribution of the rainfall intensity, as well as differences in roof-top and other surface retentions.

In routing the runoff, the direction of the subcatchment flow is assumed perpendicular to the gutter. The overland surface (particularly the street pavement) of the Oakdale Basin, as in many American cities, is reasonably homogeneous and that only limited number of subcatchment strips need to be computed and other strips can simply use the result obtained. This simplification considerably reduces the computer time and costs without sacrificing the accuracy of the results. The initial condition for the overland flow as well as for gutter flow is a depth of 0.0012 in. or 0.03 mm of water with zero velocity.

The subcatchment runoffs together with the direct rainfall constitute the input into the gutter flow. The gutter infiltration is evaluated using Horton's formula (Eq. 34) with the same values of f , k and f_c as for the street pavement since no better information is available, although the model allows different sets of values for the pavement and gutter. The gutter flows are routed from the upstream, i.e., from the gutter with the highest elevation of the basin towards downstream, as shown by the arrows in Fig. 16. The westward flow direction of Oakdale Avenue east of Junction 118 is opposite to the direction of the slope of the sewers underneath. At the downstream end of a gutter, water flows into the inlet with a capacity given by Eqs. 63 and 64, in which the discharge coefficients C_d used are 3.0 and 0.6, respectively. Not all of the inlets are placed immediately from the curb line and some may extend beyond the gutter proper at the pavement side. Such irregularities are not accounted for in the model. Also, the circular grate inlets are approximated by rectangular inlets of equivalent area in the computation. Moreover, as mentioned in Sec. VI-3, should the inlet discharge be smaller than the gutter discharge, the excessive flow is assumed carried through the inlet continuing into the next gutter immediately following.

The outflow hydrographs obtained from the surface runoff model are then used as the input into the sewer system for routing using the ISS Model to give the drainage basin runoff hydrographs. Since the sewer junctions in the Oakdale Basin have small cross-sectional areas, they are considered as point-type junctions in the ISS Model. Also, the flow measurement flume located at the outlet of the Oakdale Basin is approximated by a 0.76-m (30-in.) diameter pipe in the calculation. The computed hydrographs for the four rainstorms tested using the Illinois Urban Storm Runoff Method with the values of infiltration parameters for subcatchments listed in Table 10 are plotted in Figs. 37 to 40 for comparison with the recorded hydrographs and computed hydrographs by other methods.

X-2. COMPARISON OF THE METHODS

The evaluation of the eight methods are made against the recorded hydrographs for the four rainstorms tested. Of course there is no guarantee on the measurement accuracy of the recorded results. In fact, the accuracy of rainfall and runoff measurements for the Oakdale Basin data has not been adequately established. For example, the sharp drop at the peak discharge of the July 7, 1964 data and the strange shape of the recorded hydrograph in the next five min appear to be somewhat suspicious. For the July 2, 1960 rainstorm the peak discharge occurred at the same instant as the end of the maximum rainfall intensity, leaving no time lag between the two, is also questionable. Nevertheless, in view of the general situation of poor quality of field data for urban storm run-offs, the Oakdale data should be considered as one of the best sets that one can utilize.

The comparison of the methods can be made from two aspects. First is the ability of the methods to reproduce the recorded hydrographs based on the recorded rainfalls. Second is the computational time and cost required for the methods. For the first, in addition to the hydrographs shown in Figs. 37 to 40, the peak discharge Q_p and its time of occurrence, t_p , for the four rainstorms tested are listed in Table 8.

As mentioned in Section IX the rational method gives only the peak discharge and hence its comparison with other methods is limited. The rational method is extremely simple and gives a reasonably accurate estimation of Q_p if the runoff coefficient, C , and the time of concentration, t_c , used for rainfall intensity determination are properly chosen. However, the choice of C and t_c is more an art of judgement than a scientific precision determination, and as customarily used neither C nor t_c takes into account the preceding surface moisture condition or the temporal and areal distributions of the rainfall intensity.

The unit hydrograph method provides reasonable though not very accurate runoff hydrographs, and it is simple, fast, and straight forward if the unit hydrograph for the drainage basin is available. Because the unit hydrograph theory itself involves the assumption of linearity (Yen et al., 1968, 1973), one cannot expect high accuracy prediction from unit hydrographs, particularly for small urban drainage basins of a few acres (hectares). From the practical viewpoint, the biggest problem of using unit hydrograph is its availability. For most urban drainage basins there are no data to establish the unit hydrographs. Occasionally, transposing the known unit hydrograph of a neighboring drainage basin of similar nature with appropriate size and other adjustments may be used. This is indeed a practical approach but one should not expect high accuracy from it.

None of the methods evaluated consistently reproduces the recorded runoff hydrographs faithfully. In general the three most sophisticated methods, namely the Illinois Urban Storm Runoff Method, the EPA SWMM,

the Dorsch method, usually give better results than the other methods. The Chicago method, TRRL method, and UCUR method may give results considerably different from the recorded hydrographs and from those predicted by the more sophisticated methods. However, a more precise, detailed, and meaningful comparison of accuracy is not possible at present because of the uncertainties involved in the magnitude and areal and temporal distributions of infiltration as discussed earlier. As shown in Fig. 43, the predicted runoff hydrographs are quite sensitive to the values of infiltration used. Although attempts had been made in this study to use the same infiltration function as much as possible, differences still exist for different rainstorms and for different methods. As pointed out by Torno (1974), infiltration of an amount different from that for other methods was used for the UCUR method for the July 2, 1960 and July 7, 1964 rainstorms. Presumably the agreement of the UCUR predictions with the recorded hydrographs would be poorer if the same infiltration is used.

Furthermore, at least for the IUSR Method and presumably also for other sophisticated methods, the accuracy on the details of the basin geometry has profound influence on the shape of the hydrograph as shown in Fig. 43a. During the investigation of the effect of infiltration on hydrographs, it was thought that since the Oakdale Basin is reasonably symmetric with respect to Oakdale Avenue, computer time and cost could be saved by computing the surface runoff for one half of the basin and then double the results for sewer routing as a first approximation before further refinement is made. The rainstorm of July 2, 1960 was tested with the actual geometry and symmetric approximation for infiltration $k = 20$, $f_c = 0.5$ mm/hr (0.02 in./hr) and $f_o = 14$ mm/hr (0.55 in./hr). The difference between the two resulted hydrographs of assumed symmetric and nonsymmetric cases are surprisingly large, 17% difference in the peak discharge. Presumably when the amount of infiltration increases the difference would decrease. Nevertheless, this result indicates that for the sophisticated models for overland flow routing the accuracy of the basin geometry is one important factor to produce reliable hydrographs.

The difficulty in obtaining reliable comparison for the different methods because of the uncertainties in infiltration suggests that further study should be done on this line and that an even smaller drainage basin than the Oakdale Basin with accurate detailed data should be used for testing, and perhaps a separate testing on the surface runoff and sewer runoff be conducted. This separate testing of surface and sewer runoffs would more positively identify the relative merit of the different methods. At present, it is intended in this report to provide from a practical viewpoint some useful information for an engineer to decide which method he should choose to use for his particular project.

As mentioned earlier, the computation time and effort for the rational method is minimal, and that for the unit hydrograph method is also small provided the unit hydrograph is available. For the other six methods the use of the computer is highly recommended, and in fact the three most

sophisticated methods cannot be done without using a computer. The relative amount of computer time for the six methods varies depending on the rainstorm and drainage basin. In addition to the experience gained in this study, some information on computer time for certain methods have been reported by Heeps and Mein (1973) and James F. MacLaren, Ltd. (1973). Roughly for a drainage basin like the Oakdale with the rainstorms similar to the four tested, the process time for TRRL and Chicago methods are of the order of a tenth of a min on the University of Illinois IBM 360/75 system, with the latter slightly longer. The UCUR method requires a computer process time more than one order of magnitude longer than the TRRL method with no obvious improvement in accuracy. The computer time for EPA SWMM is about the same as that for UCUR method and the Dorsch method is slightly more. The IUSR method requires the longest computer time among the eight methods evaluated, being about twice as much as that for the EPA SWMM, or somewhat longer than 20 min for each of the four Oakdale rainstorms tested.

The evaluation made in this as well as other similar studies on urban runoff methods is definitely of limited scope and not exhaustive, and one should interpret the results with caution. Nevertheless, it can be concluded that if only the peak discharge is required and a quick result is expected without high accuracy, the rational method is the most suitable one to use. However, if the runoff hydrograph is needed such as for flow regulation and storm runoff pollution control purposes, the rational method is unacceptable and other methods should be sought. Among the seven methods evaluated that produce runoff hydrographs, the IUSR method most likely will give the most accurate result and is recommended if high accuracy is required and no restrictions on computational costs. The drawbacks of the IUSR method are that (a) it requires a large amount of computer time and hence costs; (b) it requires detailed data on basin physical characteristics that are not required by the other methods; and (c) at present the sewer routing model (ISS Model) allows only circular pipes, and for sewers having other cross sectional shapes the equivalent pipe diameter giving similar depth-area or depth-hydraulic radius relationship should be first determined.

The EPA SWMM can be operated with much less required data on basin physical characteristics than the IUSR method. Although it may not produce consistently relatively reliable hydrographs as the latter method, it is much cheaper to use and is well documented and relatively most well known. It also has many other features that the IUSR and Dorsch methods do not have. It is recommended as an alternative useful practical method.

The Dorsch method, though less accurate than the IUSR method, also requires less computer time. Nevertheless, its data requirement is as detailed and demanding as the IUSR method; and most likely it requires more computer time than EPA SWMM with similar accuracy.

When less accurate result is acceptable but the entire hydrograph is required, the unit hydrograph method should be used whenever it is possible. It is relatively simple, cheap, and fast; and it offers an accuracy at least comparable, if not better than the Chicago, UCUR, or TRRL methods. It does not require the use of computer to give the hydrograph, although using computer would save time, and programming is rather simple. However, if synthetic means or basin transposition is used to obtain the unit hydrographs, one should be careful on the reliability and accuracy.

Should the unit hydrograph not be available for the drainage basin of interest and the required accuracy of the hydrograph is not very high, the TRRL method appears to be the next choice because its data requirement is not very high and the computer time is rather short. The Chicago method does not seem to offer anything better than the TRRL method; in fact, it often over-predicts the peak discharge, and it requires more basin data and computer time. However, one should keep in mind that the TRRL method tends to over predict when the basin has high percentage of impervious areas and under predict when the percentage of pervious area is large. The UCUR method differs little from the Chicago method and yet requires considerably more data and longer computer time, and it tends to over estimate the storm peak runoff. So it should be considered only as the last resort if other methods cannot be used.

Finally, it should be mentioned here that both the ISS Model and EPA SWMM have built-in mechanism to consider the effectiveness of using in-line storage for runoff quantity and quality control purpose should it be desired to do so. Presumably the Dorsch HVM can also consider this storage effect. The total runoff volume of course simply corresponds to the total area under the hydrograph. The shape of the hydrograph, i.e., the time distribution of the runoff, will be altered with different in-line storage. An example of alternating a storm sewer system design by including in-line storage utilizing the ISS model has been reported by Yen and Sevuk (1975).

SECTION XI

REFERENCES

- Akan, A. O. Unsteady Gutter Flow into Grate Inlets. *M.S. Thesis*, Dept. of Civil Eng., Univ. of Illinois at Urbana-Champaign, August 1973.
- American Public Works Association. Problems of Combined Sewer Facilities and Overflows. *Water Pollut. Control Res. Ser.*, No. WP-20-11, Federal Water Pollution Control Administration, December 1967.
- American Public Works Association. Water Pollution Aspects of Urban Runoff. *Water Pollut. Control Res. Ser.* No. WP-20-15, Federal Water Pollution Control Administration, January 1969.
- American Society of Civil Engineers, and Water Pollution Control Federation. Design and Construction of Sanitary and Storm Sewers. *ASCE Manual* No. 37, 1969.
- Anderson, J. J. Real-Time Computer Control of Urban Runoff. *Jour. Hyd. Div.*, ASCE 96(HY1):153-164, January 1970.
- Anderson, J. J., R. L. Callery, and D. J. Anderson. An Investigation of the Evaluation of Automation and Control Schemes for Combined Sewer Systems. *Tech. Report* No. 4, OWRR Proj. C-2207, Dept. of Civil Eng., Colorado State Univ., Fort Collins, Colorado, January 1972.
- California Department of Public Works, Division of Highways. California Culvert Practice. 2nd ed., 1953.
- Chow, V. T. Frequency Analysis of Hydrologic Data with Special Application to Rainfall Intensities. *Eng. Expt. Sta. Bull.* No. 414, Univ. of Illinois at Urbana-Champaign, Ill., 1953.
- Chow, V. T. *Open-Channel Hydraulics*. McGraw-Hill Book Co., New York, 1959.
- Chow, V. T. Hydrologic Determination of Waterway Areas for the Design of Drainage Structures in Small Drainage Basins. *Eng. Expt. Sta. Bull.* No. 462, Univ. of Illinois at Urbana-Champaign, Ill., 1962.
- Chow, V. T., ed. *Handbook of Applied Hydrology*. McGraw-Hill Book Co., New York, 1964.
- Eagleson, P. S. *Dynamic Hydrology*. McGraw-Hill Book Co., New York, 1970.

Engineering-Science, Inc. Characterization and Treatment of Combined Sewer Overflows. *Report* submitted by the City and County of San Francisco Dept. of Public Works to the Federal Water Pollution Control Administration, November 1967.

Envirogenics Company (Division of Aerojet-General Corp.). Urban Storm Runoff and Combined Sewer Overflow Pollution. *Water Pollut. Control Res. Ser. No. 11024 FKM 12/71*, U.S. Environmental Protection Agency, December 1971.

Field, R., and E. J. Struzeski, Jr. Management and Control of Combined Sewer Overflows. *Jour. Water Poll. Cont. Fed.*, 44(7):1393-1415, July 1972.

Field, R., and P. Szeleley Urban Runoff and Combined Sewer Overflow (literature review). *Jour. Water Pollut. Control Fed.*, 46(6):1209-1226, 1974.

Field, R., and P. Weigel. Urban Runoff and Combined Sewer Overflow (literature review). *Jour. Water Pollut. Control Fed.*, 45(6):1108-1115, 1973.

Grayman, W. M., and P. S. Eagleson. Streamflow Record Length for Modeling Catchment Dynamics. *Report No. 114*, Hydrodynamics Lab., MIT, Cambridge, Mass., 1969.

Hawkins, R. H., and J. H. Judd. Water Pollution as Affected by Street Salting. *Water Resources Bull.*, 8(6):1246-1252, December 1972.

Heaney, J. P., W. C. Huber, H. Sheikh, J. R. Doyle, and J. E. Darling Storm Water Management Model: Refinements, Testing and Decision-Making. *Report*, Dept. of Environmental Eng. Sci., University of Florida, Gainesville, Florida, June 1973.

Heaps, D. P., and R. G. Mein. An Independent Evaluation of Three Urban Stormwater Models. *Civil Eng. Research Report No. 4/1973*, Monash Univ., Clayton, Vic., Australia, 1973.

Heaps, D. P., and R. G. Mein. Independent Comparison of Three Urban Runoff Models. *Jour. Hyd. Div., ASCE*, 100(HY7):995-1009, July 1974.

Horton, R. E. The Interpretation and Application of Runoff Plot Experiments, with Reference to Soil Erosion Problems. *Proc. Soil Sci. Soc. Am.* 3:340-349, 1938.

Huber, W. C., J. P. Heaney, M. A. Medina, W. A. Peltz, H. Sheikh, and G. F. Smith. Storm Water Management Model User's Manual Version II. *Environmental Protection Technology Ser. No. EPA-670/2-75-017*, U.S. Environmental Protection Agency, 1975.

Izzard, C. F. Hydraulics of Runoff from Developed Surfaces. *Proc. Highway Res. Board*, 26:129-146, 1946.

James F. MacLaren, Ltd. Review of Canadian Storm Sewer Design Practice and Comparison of Urban Hydrologic Models. Unpublished Draft Report for Canadian Centre for Inland Waters, 1973.

Jens, S. W. Drainage of Airport Surfaces -- Some Basic Design Considerations. *Trans. ASCE*, 113:785-809, 1948.

Kerby, W. S. Time of Concentration for Overland Flow. *Civil Engineering*, 29(3):174 (issue p. 60), March 1959.

Klym, H., W. Koiger, F. Mevius, and G. Vogel. Urban Hydrological Processes. (Paper Presented in the Seminar *Computer Methods in Hydraulics* at the Swiss Federal Institute of Technology, Zurich, Switzerland). Dorsch Consult, Munich, Germany, February 17, 1972.

Linsley, R. K., M. A. Kohler, and J. L. H. Paulhus. *Applied Hydrology*, McGraw-Hill Book Co., New York, 1949.

McPherson, M. B. Some Notes on the Rational Method of Storm Drain Design. *Tech Memo*. No. 6, ASCE Urban Water Resources Program, January, 1969.

Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc. Storm Water Management Model. *Water Poll. Cont. Res. Ser.* No. 11024 DOC, Vol. 1-4, U.S. Environmental Protection Agency, 1971.

Papadakis, C. N., and H. C. Preul. University of Cincinnati Urban Run-off Model. *Jour. Hyd. Div., ASCE*, 98(HY10):1789-1804, October, 1972.

Philip, J. R. Theory of Infiltration. In: *Advances in Hydrosociences*, ed. by V. T. Chow, 5:215-296, 1969.

Poetner, H. G. Existing Automation, Control and Intelligence Systems for Metropolitan Water Facilities. *Tech. Report* No. 1, OWRR Proj. C-2207, Dept. of Civil Eng., Colorado State Univ., Fort Collins, Colorado, January, 1972.

Potter, W. P. Peak Rates of Runoff from Small Watersheds. *BPR Bull. Hyd. Design Ser.*, No. 2, April 1961.

Price, R. K. Comparison of Four Numerical Methods for Flood Routing. *Jour. Hyd. Div., ASCE*, 100(HY7):879-899, July, 1974.

Sartor, J. D., and G. B. Boyd. Water Pollution Aspects of Street Surface Contaminants. *Environmental Protection Technology Ser.* No. EPA-R2-72-081, U.S. Environmental Protection Agency, November 1972.

- Sevuk, A. S. Unsteady Flow in Sewer Networks. *Ph.D. Thesis*, Dept. of Civil Eng., Univ. of Illinois at Urbana-Champaign, Illinois, 1973.
- Sevuk, A. S., and B. C. Yen. A Comparative Study on Flood Routing Computation. *Proc. Internat. Symp. on River Mechanics*, 3:275-290, Bangkok, January 1973.
- Sevuk, A. S., B. C. Yen, and G. E. Peterson. Illinois Storm Sewer System Simulation Model: User's Manual. *Research Report No. 73*, Water Resources Center, Univ. of Illinois at Urbana-Champaign, Ill., October 1973; (available from U.S. NTIS, Springfield, Va., as PB 227 338).
- Shen, H. W., and R. M. Li. Rainfall Effect on Sheet Flow over Smooth Surface. *Jour. Hyd. Div., ASCE*, 99(HY5):771-792, May 1973.
- Shen, Y. Y., B. C. Yen, and V. T. Chow. Experimental Investigation of Watershed Surface Runoff. *Civil Eng. Studies Hyd. Eng. Ser. No. 29*, Univ. of Illinois at Urbana-Champaign, Illinois, September 1974.
- Sherman, L. K. Streamflow from Rainfall by the Unit-Graph Method. *Eng. News-Rec.*, 108:501-505, April 7, 1932.
- Terstriep, M. L., and J. B. Stall. Urban Runoff by Road Research Laboratory Method. *Jour. Hyd. Div., ASCE*, 95(HY6):1809-1834, November 1969.
- Terstriep, M. L., and J. B. Stall. The Illinois Urban Drainage Area Simulator, ILLUDAS. *Bulletin 58*, Illinois State Water Survey, Champaign, Illinois, 1974.
- Tholin, A. L., and C. J. Kiefer. The Hydrology of Urban Runoff. *Trans. ASCE*, 125:1308-1379, 1960.
- Torno, H. Discussion of: Methods for Determination of Urban Runoff by C. N. Papakakis and H. C. Preul. *Jour. Hyd. Div., ASCE*, 100(HY8):1179-1180, August 1974.
- Tucker, L. S. Oakdale Gaging Installation, Chicago - Instrumentation and Data. *Tech. Memo. No. 2*, ASCE Urban Water Resources Research Program, 1968; (available from NTIS as PB 182 787).
- Tucker, L. S. Control of Combined Sewer Overflows in Minneapolis-St. Paul. *Tech. Report No. 3*, OWRR Proj. C-2207, Dept. of Civil Eng., Colorado State Univ., Fort Collins, Colorado, October 1971.
- U.S. Army Corps of Engineers. Runoff from Snowmelt. *Engineering and Design Manuals EM 1110-2-1406*, January 5, 1960.
- U.S. National Weather Service. Rainfall Frequency Atlas of the United States. *Weather Bureau Tech Paper No. 40*, 1961.

U.S. Soil Conservation Service. *SCS National Engineering Handbook*, Section 4, Hydrology. p. 10.1-10.24, January 1971.

University of Cincinnati Department of Civil Engineering. Urban Runoff Characteristics. *Water Pollut. Control Res. Ser. No. 11024 DQU*, U.S. Environmental Protection Agency, October 1970.

Vogel, G., and H. Klym. Die Ganglinien-Volumen-Methode. (Paper presented at the *Workshop on Methods of Sewer Network Calculation*, Dortmund, Germany). Dorsch Consult, Munich, Germany, December, 1973.

Watkins, L. H. The Design of Urban Sewer Systems. *Road Research Technical Paper No. 55*, Dept. of Sci. and Ind. Research, London, 1962.

Weibel, S. R., R. J. Anderson, and R. L. Woodward. Urban Land Runoff as a Factor in Stream Pollution. *Jour. Water Pollut. Control Fed.*, 36:914-929, 1964.

Yen, B. C. Methodologies for Flow Prediction in Urban Storm Drainage Systems. *Research Report No. 72*, Water Resources Center, Univ. of Illinois at Urbana-Champaign, Illinois, August 1973a; (available from U.S. NTIS, Springfield, Va. as PB 225 480).

Yen, B. C. Open-Channel Flow Equations Revisited. *Jour. Eng. Mech. Div., ASCE*, 99(EM5):979-1009, October 1973b.

Yen, B. C. Open-Channel Roughness -- A Review of Manning's Roughness Factor. *Sanderforschungsbereich 80 Publication SFB 80/T/57*, Karlsruhe University, Germany, 1975.

Yen, B. C., B. L. Anderson, and V. T. Chow. On the Validity of Unit Hydrograph Theory. (Paper presented at the 50th Annual Meeting of Am. Geophys. Union, Washington, D.C., April 22, 1969.)

Yen, B. C., and A. H.-S. Ang. Risks Analysis in Design of Hydraulic Projects. *Stochastic Hydraulics* (Proc. Internat. Symp. Stochastic Hydraulics), Pittsburgh, p. 694-709, 1971.

Yen, B. C., V. T. Chow, and Y. Y. Shen. Further Study on the Validity of Unit Hydrograph Theory. (Paper presented at the 1973 Fall Annual Meeting of Am. Geophys. Union, San Francisco, December 12, 1973.)

Yen, B. C., and A. S. Sevuk. Design of Storm Sewer Networks. *Jour. Environ. Eng. Div., ASCE*, 101(EE4):535-553, August 1975.

Yen, B. C., W. H. Tang, and L. W. Mays. Designing Storm Sewers Using the Rational Method. *Water & Sewage Works*, 121(10):92-95, 121(11):84-85, 1974.

Yen, B. C., H. G. Wenzel, Jr., and Y. N. Yoon. Resistance Coefficients for Steady Spatially Varied Flow. *Jour. Hyd. Div., ASCE*, 98(HY8):1395-1410, August 1972.

SECTION XII
NOTATION AND ABBREVIATIONS

NOTATION

- A = area;
- A_s = water surface area of overland, gutter, sewer, or junction;
- B = parameter for exponential density function (Eq. 20) or gamma density function (Eq. 21);
- b = water surface width;
- C = parameter for gamma density function (Eq. 21); also, constant; also, runoff coefficient;
- C_d = discharge coefficient;
- c = Chezy's roughness factor; also, concentration;
- D = rainfall depth; also, diameter; also, hydraulic depth = A/b ;
- d = detention storage;
- $\bar{d}$ = average rainfall depth per time interval (Eq. 3);
- d_j = depth of rainfall of the j-th time interval;
- $E(x)$ = expected value of x;
- F = cumulative infiltration expressed in depth;
- f = infiltration capacity; also, Weisbach resistance coefficient;
- f_c = final infiltration capacity;
- f_o = initial infiltration capacity;
- G = second moment arm of hyetograph (Eq. 5); also, a function of flow depth, = $\partial Q / \partial h$;
- g = gravitational acceleration;
- H = available head of flow;
- h = flow depth;

I = inflow rate;
 i = rainfall intensity;
 j = index number;
 k = decay rate of infiltration in Horton's formula; also, surface roughness;
 L = length
 M = daily snow melt;
 n = Manning's roughness factor; also, a number;
 P = cumulative rainfall in depth;
 P_r = daily rainfall;
 Q = discharge;
 Q_p = peak discharge;
 q = lateral inflow per unit length of circumference σ ;
 $q_l = \int_{\sigma} q d\sigma$, lateral discharge per unit length of flow;
 R = hydraulic radius;
 $Re = VR/\nu$, Reynolds number;
 S = slope;
 S_f = friction slope;
 $S_o = \sin\theta$, bed slope;
 s = depression storage supply rate; also storage;
 s_c = depression storage capacity expressed in depth;
 T_a = mean daily temperature of saturated air at 10-ft level;
 t = time;
 $\bar{t}$ = first moment arm of hyetograph (Eq. 4);
 t_b = elapsed time between the end of a rainstorm and the beginning of the following rainstorm;

t_d = rainfall duration;
 U_1 = x-component of velocity of lateral flow;
 V = flow velocity;
 $V(x)$ = variance of x ;
 W = gutter width;
 x = longitudinal direction;
 z = elevation above horizontal reference datum;
 θ = angle;
 ν = kinematic viscosity of water;
 σ = perimeter bounding flow area A ; and
 σ_d = standard deviation

ABBREVIATIONS

ac - acres
ASCE - American Society of Civil Engineers
cfs - cubic feet per second
Col. - Column
EPA - U.S. Environmental Protection Agency
fps - feet per second
ft - feet
hr - hours
in. - inches
in./hr - inches per hour
ISS - Illinois Storm Sewer System Simulation (Model)
IUSR - Illinois Urban Storm Runoff (Method)

JCL - job control language
 K - thousand
 km - kilometers
 km^2 - square kilometers
 $\ln$ - nature logarithm
 $\log$ - $\log_{10}$
 LST - local standard time
 Max - maximum
 m - meters
 m/sec - meters per second
 m^2 - square meters
 m^3/sec - cubic meters per second
 mg/l - milligrams per liter
 mi - miles
 Min, min - Minimum, or minutes
 mm - millimeters
 mm/hr - millimeters per hour
 No. - number
 ppm - parts per million
 sec - seconds
 SI - International System Units
 sq ft - square feet
 sq m - square meters
 sq mi - square miles
 SWMM - Storm Water Management Model

TRRL - Transport and Road Research Laboratory (Method)
UCUR - University of Cincinnati Urban Runoff (Model)
US NCC - U.S. National Climatic Center
WPCF - Water Pollution Control Federation
yr - years

APPENDICES

	Page
A. Description and Listing of Computer Program for Frequency Analysis of Hourly Precipitation Data	142
(a) General Information	142
(b) Program Description and Input Requirements	146
(c) Subroutine Descriptions	160
(d) Dimension Requirements	160
(e) Printed Output	161
(f) Listing of Computer Program	167
B. Data Preparation and Listing of Computer Program for Illinois Surface Runoff Model	187
(a) Data Preparation	187
(b) Printed Output	196
(c) Program Listing	201
C. Data Preparation and Listing of Computer Program for Illinois Sewer System Water Quality Model	217
(a) Data Preparation	217
(b) Printed Output	222
(c) Program Listing	225
D. Application of Illinois Urban Storm Runoff Method to a Hypothetical Basin	229
(a) Procedure of Model Implementation	229
(b) Hypothetical Drainage Basin	231
(c) Hyetograph for the Hypothetical Basin	231
(d) Computation of Surface Runoff	235
(e) Computation of Flow in Sewer System	235

APPENDIX A

DESCRIPTION AND LISTING OF COMPUTER PROGRAM FOR FREQUENCY ANALYSIS OF HOURLY PRECIPITATION DATA

(a) General Information

The computer program for the frequency analysis of hourly precipitation data is written in Fortran IV language. The program in its present form requires about 160K bytes of storage. The computer time required for the analyses described in Sec. IV-1 is about 90 sec on an IBM System 360/75. The storage and time requirements may vary considerably depending on the particular application. The execution of computer program does not require any special setups.

The frequency analysis program consists of four interrelated sub-programs (PROGRAM 1, PROGRAM 2, PROGRAM 3, PROGRAM 4) and three sub-routines (SORT, TAB1, TAB2) which will be described in Subsections (b) and (c). A block diagram showing the connections between the sub-programs is given in Fig. A-1.

The program can handle either English or SI (International System) units. All inputs are either in English units or in SI units; combinations are not allowed. If inputs are in English units, the outputs are printed in English units and, if desired, also in SI units. If inputs are in SI units, the outputs are printed only in SI units.

In the program, the rainstorm parameters are referred to by their numbers which are as follows:

Parameter No.	Description
1	Time between the end of the previous rainstorm and the beginning of the current rainstorm, t_b , in hr
2	Rainstorm duration, t_d , in hr
3	Total depth of rainstorm, D (Eq. 1), in mm or in.
4	Average intensity of rainstorm, i , in mm/hr or in./hr
5	Standard deviation of rainstorm depth, σ_d (Eq. 2), in mm or in.

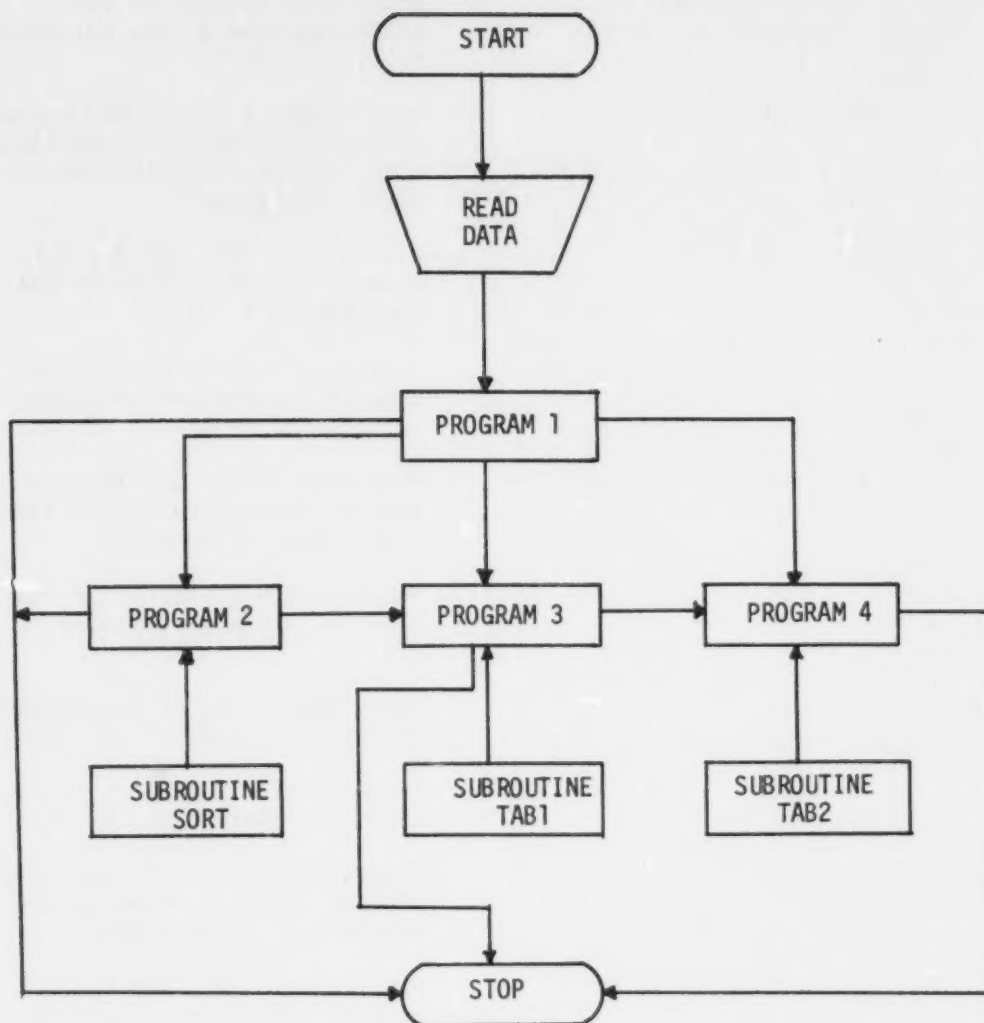


Fig. A-1. Block diagram for frequency analysis computer program

Parameter No.	Description
6	First moment arm of the hyetograph with respect to the beginning time of the rainstorm, $\bar{t}$ (Eq. 4), in hr
7	Second moment arm of the hyetograph with respect to the beginning time of the rainstorm, G (Eq. 5), in hr^2
8	Dimension a (Eq. 14) for triangular representation of the hyetograph, in hr
9	Dimension b (Eq. 15) for triangular representation of the hyetograph, in hr
10	Dimension h (Eq. 16) for triangular representation of the hyetograph, in mm or in.
11	Nondimensional standard deviation of rainstorm depth, σ_d^0 (Eq. 7)
12	First moment arm of the nondimensional hyetograph, $\bar{t}^0$ (Eq. 8)
13	Second moment arm of the nondimensional hyetograph, G^0 (Eq. 9)
14	Dimension a^0 (Eq. 17) for triangular representation of the nondimensional hyetograph
15	Dimension b^0 (Eq. 18) for triangular representation of the nondimensional hyetograph
16	Dimension h^0 (Eq. 19) for triangular representation of the nondimensional hyetograph

The data deck for the frequency analysis program should follow the source program deck which consists of subprograms PROGRAM 1, PROGRAM 2, PROGRAM 3, PROGRAM 4 and subroutines SORT, TAB1, TAB2 (Fig. A-2). The data decks for PROGRAM 1, PROGRAM 2, PROGRAM 3 and PROGRAM 4 will be

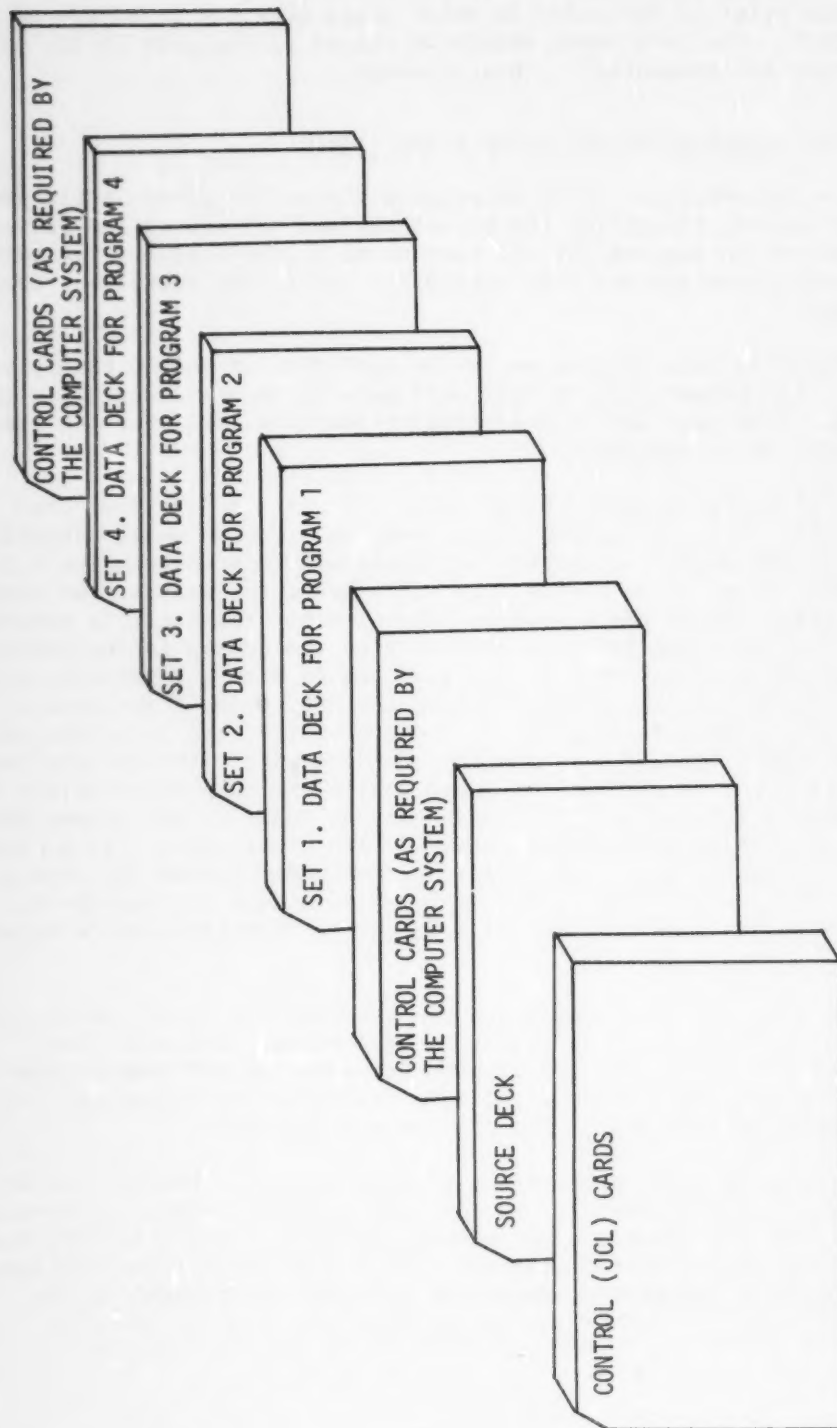


Fig. A-2. Arrangement of card decks for frequency analysis computer program

referred to in the following as Sets 1, 2, 3 and 4, respectively. Set numbers also refer to the order in which these sets are to be placed in the data deck. The data cards should be placed in the sets in the same order as they are presented in this appendix.

(b) Program Descriptions and Input Requirements

PROGRAM 1 — Description: This subprogram traces the given hourly precipitation record, identifies the rainstorms and calculates the values of 16 rainstorm parameters for all rainstorms in the record. The values of rainstorm parameters are then stored for use in the subsequent three subprograms.

The main input to this subprogram is the card deck of hourly precipitation data. The preparation of this card deck is explained later in this subsection. This card deck should contain only the cards for the season and the years to be analyzed.

The length of a season should be at least one month and no more than twelve months. The same season of the same consecutive months should be used for all the years. A season can extend over two consecutive calendar years. If a season begins with a dry hour, the elapse time between the first rainstorm in the season and the previous rainstorm is assumed to be equal to the time from the beginning of the season to the beginning of the first rainstorm. If a season begins with a rainstorm, even if the rainstorm actually started before the beginning of the season, only the part of rain falling within the season is taken into consideration. For such a case, the elapse time between this rainstorm and the preceding one, t_p , is assigned as zero, and in the computer printout a minimum elapsed time of zero indicates that at least in one season among the years of data that the season started with a rainstorm. If a season ends with a rainstorm, even if it actually extended beyond the season, again only the part of rain falling within the season is considered. However, there is no message in the printout to identify such an occurrence.

The outputs from this subprogram are the printouts of hourly precipitation values and the values of rainstorm parameters. For each year, hourly precipitation values are printed for each day and hour of the season followed by the values of the sixteen rainstorm parameters for all rainstorms in that year. The printouts are optional.

PROGRAM 1 — Input: The preparation of data cards for PROGRAM 1 (Set 1) is explained in the following. In the card identification, the numeral before the dash indicates the set number and the numeral after the dash represents the card number in the set. If a card has a letter following immediately after the second numeral in the card identification, it

indicates that the amount of data requires more than one card to account for all the data. These different cards having the same identification numerals but different letters all have the same format as the first data card designated by the letter a.

Card Set 1 consists of two subsets. The first subset contains general information. The number of cards in this subset varies between 7 and 18 depending on the number of months in the season, and they are described as follows:

Card	Column	Format	Entry
1-1	1-4	I4	Dimension of array X, see Subsection (d); e.g., 2500
1-1	5-8	I4	Row dimension of array PA, see Subsection (d); e.g., 500
1-1	9-80	Blank	
1-2	1	I1	=1 if inputs are in English units =2 if inputs are in SI units
1-2	2-80	Blank	
1-3	1	I1	=0 if inputs and printouts are in identical units, (i.e., SI-SI or English-English) =1 if inputs are in English units and outputs are to be printed both in English and SI units
1-3	2-80	Blank	
1-4	1-2	I2	Number of months in the season (01-12); e.g., 3
1-4	3-80	Blank	
1-5a	1-2	I2	Number representing the first month of the season (01-12 = Jan-Dec); e.g., 06 = June
1-5a	3-80	Blank	
1-5b	1-2	I2	Number representing the second month of the season
1-5b	3-80	Blank	
...	...	...	Repeat as Card 1-5a for the succeeding months, one card for each month in the season. The total number of cards is n, where n is equal to the number in Col. 1-2 in Card 1-4; maximum value of n is 12

Card	Column	Format	Entry
1-5n	1-2	I2	Number representing the last month of the season; e.g., 08 = August
1-5n	3-80	Blank	
1-6	1-2	I2	Number of years in the record; e.g., 14
1-6	3-4	I2	Last two digits of the last year in the record; e.g., 72=1972
1-6	5-80	Blank	
1-7	1	I1	=1 if, for each year, hourly precipitation values are to be printed out for each day and hour of the season followed by the values of rainstorm parameters for all rainstorms in that year =0 otherwise
1-7	2-80	Blank	

An example of the input card subset 1 is given in Fig. A-3a.

The second subset of the card Set 1 contains the card deck of hourly precipitation data. This subset contains as many pairs of cards as the number of days with precipitation (excluding trace) plus the number of the first day of the month without precipitation in the record considered. The data cards should be placed in chronological order. For each day with precipitation, two cards are prepared as follows:

Card No. 1

Column	Format	Entry
1-6		Leave blank or use for station identification purposes. In U.S. National Climatic Center (US-NCC) format, Col. 1-2 are used to punch the state code (e.g., 11 for Illinois) and Col. 3-6 are for the station number (e.g., 8740 for Urbana). The program does not read these columns
7-8	I2	Last two digits of the year; e.g., 59 = 1959
9-10	I2	Month number (01-12 = Jan-Dec); e.g., 06 = June
11-12	I2	Day of the month (01-31); e.g., 11=eleventh day of the month

Column	Format	Entry
13	I1	Card number (=1)
14-16	I3	Precipitation amount for the hour ending 01 local standard time (LST), i.e. 1 A.M. The values are punched as integers to 0.01 in. in English units or to 0.1 mm in SI units (021 = 0.21 in. or 2.1 mm). For no precipitation, or trace, that is less than 0.01 in. for English units, or 0.1 mm for SI units, leave these spaces blank or punch "-BB," where "B" indicates a blank. The latter is US-NCC practice. The program reads these columns with F3.2 format if English units are used or with F3.1 format if SI units are used
17-19	I3	The same as Col. 14-16 except it is for the hour ending 02 LST
...	...	Repeating Col. 14-16 in I3 format for the succeeding hours
47-49	I3	The same as Col. 14-16 for the hour ending 12 LST
50-80	Blank	

Card No. 2

Column	Format	Entry
1-6		Same as in Card No. 1
7-8	I2	Same as in Card No. 1
9-10	I2	Same as in Card No. 1
11-12	I2	Same as in Card No. 1
13	I1	Card number (=2)
14-16	I3	The same as Col. 14-16 of Card No. 1 except for the hour ending 13 LST
17-19	I3	The same as Col. 14-16 of Card No. 1 except for the hour ending 14 LST
...	...	Repeat as Col. 14-16 in I3 format for the succeeding hours
47-49	I3	The same as Col. 14-16 of Card No. 1 for the hour ending 24 LST

Column	Format	Entry
50-53		Leave blank or punch daily total. The latter is US-NCC practice; punching in a manner similar to Col. 14-16 of Card No. 1 except in I4 format instead of I3; e.g., 0108 = 1.08 in. or 10.8 mm. The program does not read these columns
54-57		Leave blank or punch monthly total only on the last day of the month with precipitation. The latter is US-NCC practice; punching in a manner similar to Col. 50-53 of this card; e.g., 0109 = 1.09 in. or 10.9 mm. The program does not read these columns
58-78	Blank	
79-80	I2	Next day with precipitation. For the last day of the month with precipitation, this is "01"; e.g., 30 = next day with precipitation is the 30th day of the same month

For the first day of each month, the pair of cards just described should always be punched no matter whether there is precipitation or not. If there is no precipitation for that day, the daily total can be punched as "-BBB" or leave blank; and all the hourly precipitation as "-BB" or leave blank. If there is no precipitation for that month, monthly total can be punched as "-BBB" or leave blank. Punching "-BBB" and "-BB" is the US-NCC practice.

The card deck of hourly precipitation data supplied by US-NCC can be used directly in this frequency analysis computer program. However, it should first be checked to correct mistakes, fill in missing information, and discard unneeded data. The card deck of hourly precipitation data should contain only the cards for the season and the years to be analyzed. The length of a season is at least 1 month and at most 12 months. For any year if there is missing data within the season, the data of that year should be completely removed from the deck unless the missing information can be filled in.

An example of three pairs of cards of Subset 2 representing the data for the entire month of June, 1959, for the station at Urbana, Illinois is shown in Fig. A-3b.

PROGRAM 2 - Description: This subprogram performs the following optional operations:

- Operation 2-1: Prints the values of rainstorm parameters for all the rainstorms in the record.
- Operation 2-2: Sorts the values of each of the rainstorm parameters in an ascending order and prints the results.
- Operation 2-3: For any given rainstorm parameter, sorts the values of that parameter and prints the results together with the corresponding values of all other rainstorm parameters.
- Operation 2-4: For any given rainstorm parameter, sorts a given subset of values of that parameter and prints the results together with the corresponding values of all other rainstorm parameters.

PROGRAM 2 - Input: The number of cards in Set 2 for PROGRAM 2 is at least 1, and up to a maximum equal to the sum of 23 and the number of times that Operation 2-4 is to be performed. Operation 2-4 can be performed as many times as determined by the rainfall parameters considered and the upper and lower limits of the values of the parameters selected. The preparation of data card Set 2 is explained as follows.

Card	Column	Format	Entry
2-1	1	I1	=1 if execution of PROGRAM 2 is desired =0 otherwise. In this case no other cards should be used for PROGRAM 2
2-1	2-80	Blank	
2-2	1	I1	=1 if Operation 2-1 is to be performed =0 otherwise
2-2	2-80	Blank	
2-3	1	I1	=1 if Operation 2-2 is to be performed =0 otherwise
2-3	2-80	Blank	

Card	Column	Format	Entry
2-4	1	I1	=1 if Operation 2-3 is to be performed =0 otherwise. In this case discard Cards 2-5 and 2-6 and go directly to Card 2-7
2-4	2-80	Blank	
2-5	1-2	I2	Number of times Operation 2-3 is to be performed, maximum 16; e.g., 1
2-5	3-80	Blank	
2-6a	1-2	I2	The number of the rainstorm parameter for which Operation 2-3 will be performed; e.g., 2
2-6a	3-80	Blank	
...	...	...	If Operation 2-3 is to be performed for more than one rainstorm parameter, use a separate card as Card 2-6a for each parameter. The total number of cards is n, where n is equal to the number in Col. 1-2 of Card 2-5
2-6n	1-2	I2	
2-6n	3-80	Blank	
2-7	1	I1	=1 if Operation 2-4 is to be performed =0 otherwise. In this case discard Cards 2-8 and 2-9, and Card 2-7 becomes the last card of the data deck for PROGRAM 2
2-7	2-80	Blank	
2-8	1-2	I2	Number of times Operation 2-4 is to be performed; e.g., 2
2-8	3-80	Blank	
2-9a	1-2	I2	The number of the rainstorm parameter for which Operation 2-4 will be performed; e.g., 2
2-9a	11-20	F10.0	The lower limit for the values of the parameter to be considered; e.g., 4.0
2-9a	21-30	F10.0	The upper limit for the values of the parameter to be considered; e.g., 6.0. Only those values which are greater than or equal to the lower limit but less than the upper limit are considered.

Card	Column	Format	Entry
2-9a	31-80	Blank	
...	...	...	Repeat using a separate card as Card 2-9a for each performance of Operation 2-4. The total number of cards is n, where n is equal to the number in Col. 1-2 of Card 2-8
2-9n	1-2	I2	See Col. 1-2 of Card 2-9a; e.g., 3
2-9n	11-20	F10.0	See Col. 11-20 of Card 2-9a; e.g., 0.35
2-9n	21-30	F10.0	See Col. 21-30 of Card 2-9a; e.g., 0.65
2-9n	31-80	Blank	

An example of data card Set 2 is given in Fig. A-4.

PROGRAM 3 - Description: This subprogram performs the following optional operations of one-way and conditional frequency analysis discussed in Sec. IV-1:

- Operation 3-1: Performs one-way frequency analysis on all values of any given rainstorm parameter.
- Operation 3-2: Performs one-way frequency analysis on a subset of values of any given rainstorm parameter. This subset contains only the values which are conditional on a given range of values of another rainstorm parameter.

The outputs of the above operations are tables of frequencies (number of observations over given class intervals), relative frequencies (frequency divided by the total number of observations), probability densities (relative frequency divided by the interval size), and nonexceedance probabilities (cumulative relative frequency). In addition, minimum, maximum, mean and standard deviation of the values considered are also printed out.

PROGRAM 3 - Input: The number of cards in Set 3 for PROGRAM 3 is at least 1, and up to a maximum equal to the sum of 5 and the number of times that Operations 3-1 and 3-2 are to be performed. The preparation of data card Set 3 is explained as follows:

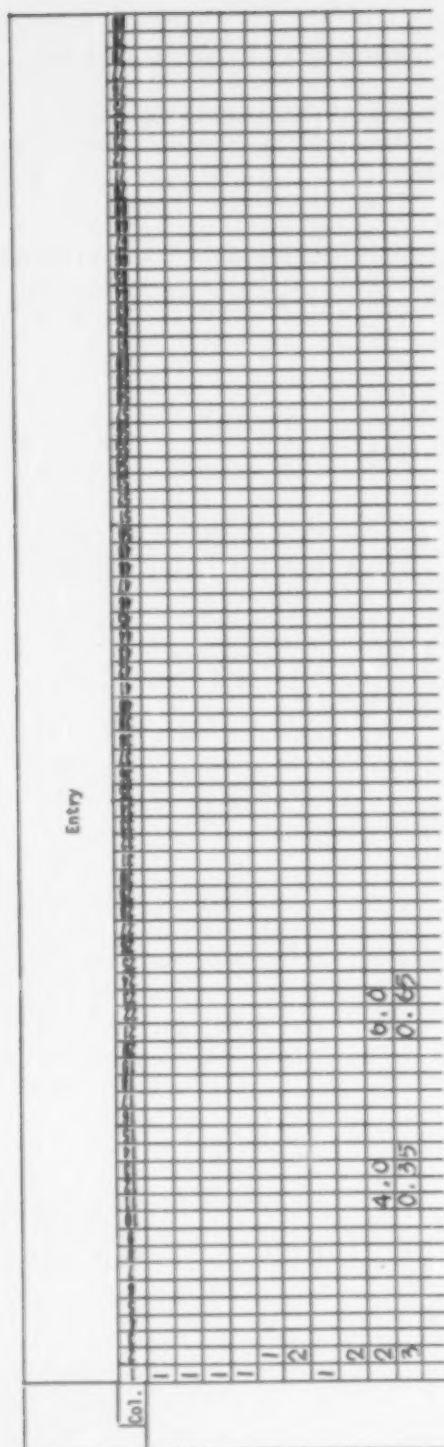


Fig. A-4. An example of data card Set 2 for frequency analysis computer program

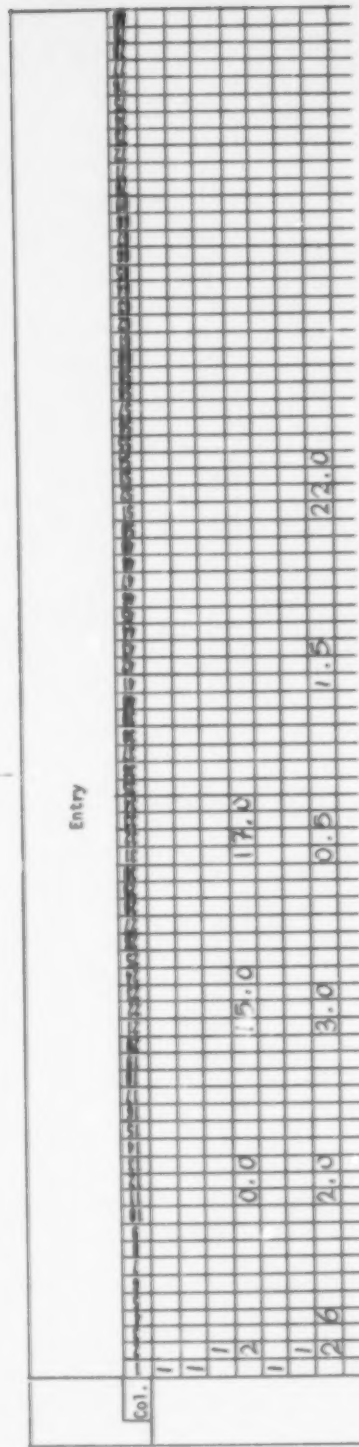


Fig. A-5. An example of data card Set 3 for frequency analysis computer program

Card	Column	Format	Entry
3-1	1	I1	=1 if execution of PROGRAM 3 is desired =0 otherwise. In this case no other data cards should be used for PROGRAM 3
3-1	2-80	Blank	
3-2	1	I1	=1 if Operation 3-1 is to be executed =0 otherwise. In this case discard Cards 3-3 and 3-4 and go directly to Card 3-5
3-2	2-80	Blank	
3-3	1-2	I2	Number of times Operation 3-1 is to be performed; e.g., 1
3-3	3-80	Blank	
3-4a	1-2	I2	The number of the rainstorm parameter for which Operation 3-1 will be performed; e.g., 2
3-4a	11-20	F10.0	The lower limit of the first class interval*; e.g., 0.0
3-4a	21-30	F10.0	The upper limit of the last class interval*; e.g., 15.0
3-4a	31-40	F10.0	Number of class intervals; e.g., 17.0; this must include two class intervals for the values under and above limits*
3-4a	41-80	Blank	
...	...	...	Repeat using a separate card as Card 3-4a for each performance of Operation 3-1. The total number of cards is n which should equal to the number punched in Col. 1-2 of Card 3-3
3-4n	1-2	I2	
3-4n	11-20	F10.0	
3-4n	21-30	F10.0	
3-4n	31-40	F10.0	
3-4n	41-80	Blank	

*If the lower limit is equal to the upper limit, the minimum and maximum values of the rainstorm parameter are used as the lower limit and the upper limit, respectively. Interval size is equal to (upper limit-lower limit)/ (number of class intervals - 2). A count is classified into a particular interval if the value is greater than or equal to the lower limit of that interval but less than the upper limit of the same interval. The above explanations also apply to Operation 3-2 and PROGRAM 4.

Card	Column	Format	Entry
3-5	1	I1	=1 if Operation 3-2 is to be performed =0 otherwise. In this case discard Cards 3-6 and 3-7, whereas Card 3-5 becomes the last card of the data deck for PROGRAM 3
3-5	2-80	Blank	
3-6	1-2	I2	Number of times Operation 3-2 is to be performed; e.g., 1
3-6	3-80	Blank	
3-7a	1-2	I2	The number of the first rainstorm parameter;* e.g., 2
3-7a	3-4	I2	The number of the second rainstorm parameter;* e.g., 6
3-7a	11-20	F10.0	The lower limit for the first rainstorm parameter;* e.g., 2.0
3-7a	21-30	F10.0	The upper limit for the first rainstorm parameter;* e.g., 3.0
3-7a	31-40	F10.0	The lower limit of the first class interval for one-way frequency analysis; e.g., 0.5
3-7a	41-50	F10.0	The upper limit of the last class interval for one-way frequency analysis; e.g., 1.5
3-7a	51-60	F10.0	Number of class intervals; e.g., 22.0; this must include two class intervals for the values under and above limits
3-7a	61-80	Blank	
...	...	...	Repeat using a separate card as Card 3-7a for each performance of Operation 3-2. The total number of cards is n, where n is equal to the number in Col. 1-2 of Card 3-6
3-7n	1-2	I2	
3-7n	3-4	I2	
3-7n	11-20	F10.0	
3-7n	21-30	F10.0	
3-7n	31-40	F10.0	
3-7n	41-50	F10.0	
3-7n	51-60	F10.0	
3-7n	61-80	Blank	

*A one-way frequency analysis is performed on the subset of values of the second rainstorm parameter (Col. 3-4). This subset is conditional on the values of the first rainstorm parameter (Col. 1-2) which are greater than or equal to the lower limit (Col. 11-20) but less than the upper limit (Col. 21-30).

An example of data card Set 3 is given in Fig. A-5.

PROGRAM 4 - Description: This subprogram performs the two-way frequency analysis described in Sec. IV-1(G) on the values of any given pair of rainstorm parameters.

The outputs from this subprogram are the two-way tables of frequencies, relative frequencies, and probability densities (relative frequency divided by the interval area).

The execution of PROGRAM 4 is optional.

PROGRAM 4 - Input: The number of cards in Set 4 for PROGRAM 4 is at least 1, and up to a maximum equal to the sum of 2 and twice the number of times that PROGRAM 4 is to be executed. The preparation of data card Set 4 is explained as follows:

Card	Column	Format	Entry
4-1	1	I1	=1 if execution of PROGRAM 4 is desired =0 otherwise. In this case no other data cards should be used for PROGRAM 4
4-1	2-80	Blank	
4-2	1-2	I2	Number of times PROGRAM 4 is to be executed; e.g., 1
4-2	3-80	Blank	
4-3a	1-2	I2	The number of the first rainstorm parameter to be cross-tabulated; e.g., 2
4-3a	11-20	F10.0	The lower limit of the first class interval for the first parameter; e.g., 0.0
4-3a	21-30	F10.0	The upper limit of the last class interval for the first parameter; e.g., 18.0
4-3a	31-80	Blank	
4-4a	1-2	I2	The number of the second rainstorm parameter to be cross-tabulated; e.g., 4

Card	Column	Format	Entry
4-4a	11-20	F10.0	The lower limit of the first class interval for the second parameter; e.g., 0.0
4-4a	21-30	F10.0	The upper limit of the last class interval for the second parameter; e.g., 1.8
4-4a	31-80	Blank	
...	...	...	Repeat using a separate pair of cards as Cards 4-3a and 4-4a for each execution of PROGRAM 4
4-3n	1-2	I2	
4-3n	11-20	F10.0	
4-3n	21-30	F10.0	
4-3n	31-80	Blank	
4-4n	1-2	I2	
4-4n	11-20	F10.0	
4-4n	21-30	F10.0	
4-4n	31-80	Blank	

In PROGRAM 4, number of class intervals (including two class intervals for the values under and above limits) for each rainstorm parameter is equal to 20. However, the desired interval size for each parameter can be obtained by the proper choice of upper and lower limits.

An example of data card Set 4 is given in Fig. A-6.

		Entry																	
Col.																			
1																			
2																			
4																			

Fig. A-6. An example of data card Set 4 for frequency analysis computer program

(c) Subroutine Descriptions

Detailed information about subroutines SORT, TAB1 and TAB2 is given in the subroutine listings. These subroutines perform the following operations:

SUBROUTINE SORT: Sorts the elements of an array in an ascending order and rearranges simultaneously the corresponding elements of an associated array.

SUBROUTINE TAB1: Performs one-way frequency analysis.

SUBROUTINE TAB2: Performs two-way frequency analysis.

Subroutines TAB1 and TAB2 are in IBM System/360 Scientific Subroutine Package Version III.

(d) Dimension Requirements

All arrays used in the frequency analysis computer program are dimensioned at the beginning of PROGRAM 1. Should it be desired to alter the dimension requirement already listed in the program for particular application, the user may change the dimensions of the arrays which are described as follows:

(1) Array X is used to store the hourly precipitation values for all hours of the season of a year. Its dimension should be equal to or greater than the number of hours within the season. For example, if the season is taken to be the month of January, the dimension of X should be equal to or greater than 744.

(2) Array D is used to store the hourly precipitation values for a rainstorm. Its dimension should be equal to or greater than the duration, in hours, of the longest rainstorm in the record.

(3) Array P is used to store the values of rainstorm parameters for all rainstorms in the season of a year. Its row dimension should be equal to or greater than the maximum number of rainstorms in the season of any one year. The row dimension is the first dimension of a two-dimensional array. For example, row dimension of array P(50,16) is 50.

(4) Array PA is used to store the values of rainstorm parameters for all rainstorms in the record. Its row dimension should be equal to or greater than the number of rainstorms in the record. Array PA1 should have the same dimensions as array PA.

(5) Arrays A and B, which are temporary storage arrays, should both have the same dimension as the row dimension of array PA.

(6) Arrays FREQ, PCT, DEN, CDEN and XX should all have the same dimension, and each of them should be at least one more than the maximum number of class intervals that will be used for any one of the one-way frequency analysis.

(e) Printed Output

The frequency analysis computer program prints by default the following information:

(1) Program title:

"FREQUENCY ANALYSIS OF HOURLY PRECIPITATION DATA"

(2) Number of years in the record; e.g.,

"14 YEARS OF RECORD"

(3) Numbers representing the months in the season; e.g., for the months of June, July and August,

"MONTHS

6

7

8"

(4) Definitions of sixteen rainstorm parameters.

All other printouts are optional and controlled by the user through the input cards as described in Subsection (b).

The printout from PROGRAM 1 lists the hourly precipitation values and the values of rainstorm parameters. For each year of record, the hourly precipitation values are listed for each day and hour of the season (Fig. A-7a) followed by the values of the sixteen rainstorm parameters for all rainstorms in that year (Fig. A-7b). In the part of printout entitled "HOURLY PRECIPITATION DATA", the first column gives the last two digits of the year (e.g., 65 for 1965; see Fig. A-7a), the second column gives the number representing the month of the year (e.g., 6 for June; see Fig. A-7a), and the third column gives the day of the month (e.g., 1 for the first day of the month; see Fig. A-7a). The last twenty-four columns (partially shown in Fig. A-7a for the Urbana, Illinois data) list the hourly precipitation values for the hours ending (in LST) as indicated by the column headings 1 through 24. Fig. A-7b is a typical example showing a portion of the printout of values of rainstorm parameters which is entitled "VALUES OF RAINSTORM PARAMETERS IN SI (or ENGLISH) UNITS". The columns of the printout are numbered

corresponding to the rainstorm parameters. Each row of the table in the printout corresponds to one rainstorm. The first row is for the first rainstorm in the season of the year under consideration, the second row for the second rainstorm, etc. The last row is for the last rainstorm of the season of the year under consideration. "Number of rainstorms =" shown in the printout refers to the total number for the season of the year under consideration; e.g., 31 shown in Fig. A-7b indicates that there were 31 rainstorms during the season of June, July and August for the year 1965 for the Urbana, Illinois data.

The printout from Operation 2-1 of PROGRAM 2, entitled "VALUES OF RAINSTORM PARAMETERS", consists of the values of the sixteen rainstorm parameters for all the rainstorms in the record. The columns of the printout are numbered corresponding to the rainstorm parameters. Each row of the table in the printout corresponds to one rainstorm. The first row is for the first rainstorm in the record, the second row for the second rainstorm, etc. "Number of rainstorms =" shown in the printout is the total number for the entire record; e.g., 455 for the season of June, July and August for the years 1959-1972 for the Urbana, Illinois data.

The printout from Operation 2-2 of PROGRAM 2 lists the sorted values of the sixteen rainstorm parameters. All values of each of the rainstorm parameters are sorted in an ascending order. The printout is entitled "VALUES OF RAINSTORM PARAMETERS SORTED IN ASCENDING ORDER", with the columns numbered corresponding to the rainstorm parameters. "Number of rainstorms" refers to the total number for the entire record.

The printout from Operation 2-3 of PROGRAM 2 lists the sorted values of a rainstorm parameter together with the corresponding values of all the other rainstorm parameters. All values of a given rainstorm parameter (say, rainstorm parameter no. 2) are sorted in an ascending order and the corresponding values of all the other rainstorm parameters are rearranged simultaneously. The printout is entitled "VALUES OF RAINSTORM PARAMETER (2) SORTED IN ASCENDING ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REARRANGED SIMULTANEOUSLY", and the columns are numbered corresponding to the rainstorm parameters. Each row of the table in the printout corresponds to one rainstorm. "Number of rainstorms" refers to the total number for the entire record.

The printout from Operation 2-4 of PROGRAM 2 is similar to that of Operation 2-3 with the exception that only a given subset of values (e.g., the values which are greater than or equal to 4.0 but less than 6.0) of the rainstorm parameter (say, rainstorm parameter no. 2) is sorted instead of all the values. The printout title is, e.g., "VALUES OF RAINSTORM PARAMETER 2 GREATER THAN OR EQUAL TO 4.0 AND LESS THAN 6.0 SORTED IN ASCENDING ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REARRANGED SIMULTANEOUSLY". "Number of rainstorms" refers to total number within the subset.

HOURLY PRECIPITATION DATA.

VALUES ARE IN MILLIMETERS.

			HOUR ENDING								
			1	2	3	4	. . .	21	22	23	24
65	6	1	0.0	0.0	0.0	0.0	. . .	6.9	16.3	4.3	7.6
65	6	2	2.3	0.0	0.3	0.5	. . .	0.0	0.0	0.0	0.0
65	6	3	0.0	0.0	0.0	0.0	. . .	0.0	0.0	0.0	0.0
65	6	4	0.0	0.0	0.0	0.0	. . .	0.0	0.0	0.0	0.0
65	6	5	0.0	0.0	0.0	0.0	. . .	0.0	0.0	0.0	0.0
65	6	6	0.0	0.0	0.0	0.0	. . .	0.3	0.3	0.0	0.0
65	6	7	0.0	0.0	0.0	0.0	. . .	0.0	0.0	0.0	0.0
65	6	8	0.0	0.0	0.0	0.0	. . .	0.0	0.0	0.0	0.0
.											

(a) Sample printout from PROGRAM 1,
hourly precipitation values

VALUES OF RAINSTORM PARAMETERS IN SI UNITS.

1	2	3	. . .	14	15	16
4.00	7.00	46.23	. . .	0.49	0.51	0.29
1.00	5.00	9.14	. . .	1.05	-0.05	0.40
107.00	4.00	4.06	. . .	-0.34	1.34	0.50
42.00	2.00	3.81	. . .	0.65	0.35	1.00
289.00	2.00	0.51	. . .	0.50	0.50	1.00
1.00	2.00	3.05	. . .	0.38	0.63	1.00
200.00	3.00	6.10	. . .	0.83	0.17	0.67
5.00	2.00	2.03	. . .	1.06	-0.06	1.00
57.00	2.00	53.09	. . .	0.45	0.55	1.00
.						

NUMBER OF RAINSTORMS = 31

(b) Sample printout from PROGRAM 1,
values of rainstorm parameters

Fig. A-7. Sample printout of frequency analysis computer program

(SI UNITS)

FREQUENCY ANALYSIS FOR RAINSTORM PARAMETER 2

LOWER LIMIT = 0.0 UPPER LIMIT = 15.00 NUMBER OF CLASS INTERVALS = 17. INTERVAL SIZE = 1.00
 MINIMUM = 1.00 MAXIMUM = 14.00 MEAN = 2.45 STANDARD DEVIATION = 2.08

INTERVAL	FREQ	REL FREQ	PROB DEN	NON-EXCE PROB
*****	0.0	0.0	0.0	0.0
0.0	0.0	0.0	0.0	0.0
1.00	176.0	0.3868	0.3868	0.3868
2.00	134.0	0.2945	0.2945	0.6813
3.00	64.0	0.1407	0.1407	0.8220
4.00	34.0	0.0747	0.0747	0.8967
5.00	16.0	0.0352	0.0352	0.9319
6.00	9.0	0.0198	0.0198	0.9516
7.00	5.0	0.0110	0.0110	0.9626
8.00	6.0	0.0132	0.0132	0.9758
9.00	2.0	0.0044	0.0044	0.9802
10.00	1.0	0.0022	0.0022	0.9824
11.00	1.0	0.0022	0.0022	0.9846
12.00	3.0	0.0066	0.0066	0.9912
13.00	3.0	0.0066	0.0066	0.9978
14.00	1.0	0.0022	0.0022	1.0000
15.00	0.0	0.0	0.0	1.0000

NUMBER OF RAINSTORMS = 455

Fig. A-7c. Sample printout of frequency analysis computer program, Operation 3-1

(SI UNITS)

FREQUENCY ANALYSIS FOR RAINSTORM PARAMETER 6

ONLY VALUES CORRESPONDING TO VALUES OF RAINSTORM PARAMETER 2 GREATER THAN OR EQUAL TO 2.00 AND LESS THAN 3.00 CONSIDERED

LOWER LIMIT = 0.50 UPPER LIMIT = 1.50 NUMBER OF CLASS INTERVALS = 22. INTERVAL SIZE = 0.05
 MINIMUM = 0.53 MAXIMUM = 1.48 MEAN = 0.92 STANDARD DEVIATION = 0.27

INTERVAL	FREQ	REL FREQ	PROB DEN	NON-EXCE PROB
0.50	0.0	0.0	0.0	0.0
0.55	8.0	0.0597	1.1940	0.0597
0.60	10.0	0.0746	1.4925	0.1343
0.65	12.0	0.0896	1.7910	0.2239
0.70	3.0	0.0224	0.4478	0.2463
0.75	11.0	0.0821	1.6418	0.3284
0.80	8.0	0.0597	1.1940	0.3881
0.85	8.0	0.0597	1.1940	0.4478
0.90	4.0	0.0299	0.5970	0.4776
0.95	7.0	0.0522	1.0448	0.5299
1.00	3.0	0.0224	0.4478	0.5522
1.05	19.0	0.1418	2.8358	0.6940
1.10	2.0	0.0149	0.2985	0.7090
1.15	6.0	0.0448	0.8955	0.7537
1.20	9.0	0.0672	1.3433	0.8209
1.25	2.0	0.0149	0.2985	0.8358
1.30	8.0	0.0597	1.1940	0.8555
1.35	4.0	0.0299	0.5970	0.9254
1.40	2.0	0.0149	0.2985	0.9403
1.45	4.0	0.0299	0.5970	0.9701
1.50	4.0	0.0299	0.5970	1.0000
1.50	0.0	0.0	0.0	1.0000

NUMBER OF RAINSTORMS = 134

Fig. A-7d. Sample printout of frequency analysis computer program, Operation 3-2

The printout from Operation 3-1 of PROGRAM 3 consists of the results of a one-way frequency analysis performed on all values of a given rainstorm parameter (say, rainstorm parameter no. 2). Fig. A-7c shows a sample printout from Operation 3-1 for rainstorm parameter 2 for the Urbana data. In the printout entitled "FREQUENCY ANALYSIS FOR RAINSTORM PARAMETER (2)", the first and second columns give the lower and upper limits of the class intervals, respectively. Each row of the table in the printout corresponds to one class interval. The second row of the first two columns (e.g., "0.0 1.00" in Fig. A-7c) and the next to last row (e.g., "14.00 15.00" in Fig. A-7c) define the first and the last class intervals, respectively. The first row (e.g., "***** 0.0" in Fig. A-7c) defines the class interval containing all the values which are less than the lower limit of the first class interval. The last row (e.g., "15.000 *****" in Fig. A-7c) defines the class interval containing all the values which are equal to or greater than the upper limit of the last class interval. The last four columns in the printout list the values of frequencies, relative frequencies, probability densities and nonexceedance probabilities, respectively. In addition, lower limit of the first class interval (e.g., 0.0 in Fig. A-7c), upper limit of the last class interval (e.g., 15.0 in Fig. A-7c), number of class intervals, interval size, and minimum, maximum, mean and standard deviation for all values of the rainstorm parameter under consideration are also printed out. "Number of rainstorms" refers to the total number for the entire record.

The printout from Operation 3-2 of PROGRAM 3 consists of the results of a conditional one-way frequency analysis performed on a given subset of values of a given rainstorm parameter (say, rainstorm parameter no. 6). This subset contains only the values which are conditional on a given range of values (e.g., the values which are greater than or equal to 2.0 but less than 3.0) of another rainstorm parameter (say, rainstorm parameter no. 2). Fig. A-7d shows a sample printout from Operation 3-2. "Number of rainstorms" refers to the total number within the subset. Minimum, maximum, mean and standard deviation are for the values within the subset. Explanations regarding the frequency analysis table for the printout of Operation 3-1 also apply to that of Operation 3-2.

The printout from PROGRAM 4 consists of the results of a two-way frequency analysis performed on all values of any given pair of rainstorm parameters (say, rainstorm parameters 2 and 4). The printout, entitled, e.g., "TWO-WAY FREQUENCY ANALYSIS FOR RAINSTORM PARAMETERS 2 AND 4", contains the two-way table of frequencies, relative frequencies and probability densities. In addition, the lower limit of the first class interval (e.g., 0.0 for both rainstorm parameters), the upper limit of the last class interval (e.g., 18.0 for the first rainstorm parameter and 1.8 for the second), number of class intervals (which is always equal to 20 for both rainstorm parameters) and interval size (e.g., 1.0 for the first rainstorm parameter and 0.1 for the second) are printed

out for both rainstorm parameters. In the tables of the printout, the first two columns define the class intervals for the first rainstorm parameter as described for Operation 3-1. Class intervals for the second rainstorm parameter are defined in a similar manner by column headings starting with the third column of each table. "Number of rainstorms" refers to the total number for the entire record considered.

(f) Listing of Computer Program

```

C.....
C
C   COMPUTER PROGRAM FOR FREQUENCY ANALYSIS
C   OF HOURLY PRECIPITATION DATA.
C
C   FOR ANY INFORMATION REGARDING THIS COMPUTER PROGRAM, THE USER
C   SHOULD REFER TO THE FOLLOWING SUBSECTIONS OF APPENDIX A.
C
C   SUBSECTION
C
C       (A) GENERAL INFORMATION
C       (B) PROGRAM DESCRIPTIONS AND INPUT REQUIREMENTS
C       (C) SUBROUTINE DESCRIPTIONS
C       (C) DIMENSION REQUIREMENTS
C       (E) PROGRAM OUTPUT
C
C.....
C
C   PROGRAM 1 ...
C
C       DIMENSION X(2500),D(50),P(50,16),PA(500,16),PA1(500,16),A(500)
C       DIMENSION B(500),FREQ(150),PCT(150),DEN(150),CDEN(150),XX(150)
C       DIMENSION MONTH(12),KYR(12),KMD(12),KNUMDY(12),IT(24),IP(16)
C       DIMENSION UBO(3),STAT5(5),NOV(2),UBO2(3,2),STAT1(3,20),STAT2(3,20)
C       DIMENSION XINS2(2),FREQ1(20,20),PCT1(20,20),DEN1(20,20),XX1(21)
C       DIMENSION XX2(21)
C
C   C *** INPUT 1-1
C       READ(5,57) IDIMX,IOIMPA
C       57 FORMAT(2I4)
C   C ***
C
C   C *** INPUT 1-2
C       READ(5,1) IUNIT
C       READ(5,1) IPRSI
C       1 FORMAT(I1)
C   C ***
C
C   C *** INPUT 1-3
C       READ(5,2) NUMMON
C       2 FORMAT(I2)
C       READ(5,3) (MONTH(I),I=1,NUMMON)
C       3 FORMAT(I2)
C   C ***
C
C   C *** INPUT 1-4
C       READ(5,4) NUMYR,LAITYR
C       4 FORMAT(2I2)
C   C ***

```



```

C
C *** INPUT 1-5
  READ(5,5) IPRI
  5 FORMAT(I1)
C ***
C
  WRITE(6,6)
  6 FORMAT('1')
  WRITE(6,7) NUMYR
  7 FORMAT('1'//43X,'FREQUENCY ANALYSIS OF HOURLY PRECIPITATION D
  IATA'//57X,I2,' YEARS OF RECORD'//63X,'MONTHS'//)
  WRITE(6,8) (MONTH(I),I=1,NUMYR)
  8 FORMAT(65X,I2)
  WRITE(6,6)
  WRITE(6,60)
  60 FORMAT(/1X,'RAINSTORM PARAMETERS ( ENGLISH UNITS )'//)
  WRITE(6,48)
  48 FORMAT(2X,'1.TIME BETWEEN RAINSTORMS,HR'/2X,'2.RAINSTORM DURATION,
  1HR'/2X,'3.TOTAL DEPTH OF RAINSTORM,IN'/2X,'4.AVERAGE INTENSITY OF
  2RAINSTORM,IN/HR'/2X,'5.STANDARD DEVIATION OF RAINSTORM DEPTH,IN'/2
  3X,'6.FIRST MOMENT ARM OF THE HYETOGRAPH WITH RESPECT TO THE BEGINN
  4ING TIME OF THE RAINSTORM,HR'/2X,'7.SECOND MOMENT ARM OF THE HYETO
  5GRAPH WITH RESPECT TO THE BEGINNING TIME OF THE RAINSTORM,HR.SQ.'I
  WRITE(6,49)
  49 FORMAT(2X,'8.DIMENSION A FOR TRIANGULAR REPRESENTATION OF HYETOGRA
  1PH,HR'/2X,'9.DIMENSION B OF TRIANGLE,MP'/1X,'10.DIMENSION H OF TRI
  2ANGLE,IN'/1X,'11.NONDIMENSIONAL STANDARD DEVIATION OF RAINSTORM DE
  3PTH'/1X,'12.FIRST MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,
  4'13.SECOND MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,'14.DIM
  5ENSION A FOR TRIANGULAR REPRESENTATION OF THE NONDIMENSIONAL HYETO
  6GRAPH'/1X,'15.DIMENSION B FOR TRIANGULAR REPRESENTATION OF NONDIME
  7NSIONAL HYETOGRAPH'/1X,'16.DIMENSION H FOR TRIANGULAR REPRESENTATI
  8ON OF NONDIMENSIONAL HYETOGRAPH'//)
  WRITE(6,61)
  61 FORMAT('1'//1X,'RAINSTORM PARAMETERS ( SI UNITS )'//)
  WRITE(6,38)
  38 FORMAT(2X,'1.TIME BETWEEN RAINSTORMS,HR'/2X,'2.RAINSTORM DURATION,
  1HR'/2X,'3.TOTAL DEPTH OF RAINSTORM,MM'/2X,'4.AVERAGE INTENSITY OF
  2RAINSTORM,MM/HR'/2X,'5.STANDARD DEVIATION OF RAINSTORM DEPTH,MM'/2
  3X,'6.FIRST MOMENT ARM OF THE HYETOGRAPH WITH RESPECT TO THE BEGINN
  4ING TIME OF THE RAINSTORM,HR'/2X,'7.SECOND MOMENT ARM OF THE HYETO
  5GRAPH WITH RESPECT TO THE BEGINNING TIME OF THE RAINSTORM,HR.SQ.'I
  WRITE(6,39)
  39 FORMAT(2X,'8.DIMENSION A FOR TRIANGULAR REPRESENTATION OF HYETOGRA
  1PH,HR'/2X,'9.DIMENSION B OF TRIANGLE,MP'/1X,'10.DIMENSION H OF TRI
  2ANGLE,MM'/1X,'11.NONDIMENSIONAL STANDARD DEVIATION OF RAINSTORM DE
  3PTH'/1X,'12.FIRST MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,
  4'13.SECOND MOMENT ARM OF THE NONDIMENSIONAL HYETOGRAPH'/1X,'14.DIM
  5ENSION A FOR TRIANGULAR REPRESENTATION OF THE NONDIMENSIONAL HYETO
  6GRAPH'/1X,'15.DIMENSION B FOR TRIANGULAR REPRESENTATION OF NONDIME
  7NSIONAL HYETOGRAPH'/1X,'16.DIMENSION H FOR TRIANGULAR REPRESENTATI
  8ON OF NONDIMENSIONAL HYETOGRAPH'//)
C
  MC3=0
C
  55 DO 9 I=1,IDIMX
    9 X(I)=0.0
C
    11=0
    12=0
    13=0
  10 12=11+12
    IF(12.EQ.0) GO TO 20
    IF(INXDY.EQ.1.AND.IMO.EQ.MONTH(NUMYR)) GO TO 11
  20 IF(IUNIT.EQ.2) GO TO 12

```

```

C
C *** INPUT 1-6
  READ(5,13) IYR,IMO,IDY,(X(I+12),I=1,24),INXDY
 13 FORMAT(6X,3I2,1X,12F3.2/13X,12F3.2,29X,12)
C ***
C
C      GO TO 14
C
C *** INPUT 1-6
 12 READ(5,15) IYR,IMO,IDY,(X(I+12),I=1,24),INXDY
 15 FORMAT(6X,3I2,1X,12F3.1/13X,12F3.1,29X,12)
C ***
C
 14 I1=24*(INXDY-IDY)
    IF(I1.GT.0) GO TO 10
    GO TO(16,18,16,17,16,17,16,16,17,16,17,16),IMO
 16 NUMDY=31
    GO TO 19
 17 NUMDY=30
    GO TO 19
 18 X1=IYR
    X2=IYR/4
    X3=X1/4
    IF(X2.EQ.X3) NUMDY=29
    IF(X2.NE.X3) NUMDY=28
 19 I1=(NUMDY+1-IDY)*24
    I3=I3+1
    KYR(I3)=IYR
    KMO(I3)=IMO
    KNUMDY(I3)=NUMDY
    GO TO 10
C
 11 IC=0
    LC=0
    MC=1
    NC=1
    DO 59 I=1,I2
      IF(X(I).GT.0.) GO TO 21
      IF(NC.NE.MC) GO TO 22
      IC=IC+1
      GO TO 59
 21 IF(LC.GT.0) GO TO 23
      P(MC,1)=IC
      NC=MC+1
      IC=1
 23 LC=LC+1
      D(LC)=X(I)
      IF(I.EQ.I2) GO TO 22
      GO TO 59
 22 P(MC,2)=LC
      SUM1=0.
      DO 24 J=1,LC
 24 SUM1=SUM1+D(J)
      P(MC,3)=SUM1
      P(MC,4)=P(MC,3)/P(MC,2)
      SUM2=0.
      SUM3=0.
      SUM4=0.
      DO 25 J=1,LC
      XJ=J
      SUM2=SUM2+D(J)*(XJ-0.5)
      SUM3=SUM3+D(J)*(XJ-0.5)**2
 25 SUM4=SUM4+(D(J)-P(MC,4))**2
      P(MC,5)=SQRT(SUM4/P(MC,2))

```

```

P(MC,6)=SUM2/P(MC,3)
P(MC,7)=SUM3/P(MC,3)+1./12.
P(MC,8)=3.*P(MC,6)-P(MC,2)
P(MC,9)=P(MC,2)-P(MC,8)
P(MC,10)=2.*P(MC,4)
P(MC,11)=P(MC,5)/P(MC,3)
P(MC,12)=P(MC,6)/P(MC,2)
P(MC,13)=P(MC,7)/(P(MC,2)**2)
P(MC,14)=P(MC,8)/P(MC,2)
P(MC,15)=P(MC,9)/P(MC,2)
P(MC,16)=P(MC,10)/P(MC,3)
MC=MC+1
LC=0
59 CONTINUE
C
    MC1=MC-1
    IF(MC1.EQ.0) GO TO 27
    DO 26 I=1,16
    DO 26 J=1,MC1
26   PA(MC3+J,I)=P(J,I)
    MC3=MC3+MC1
C
    DO 40 I=1,16
40   IP(I)=1
C
    IF(IPR1.EQ.0) GO TO 27
C
    I9=0
    WRITE(6,6)
    DO 29 I=1,24
29   IT(I)=1
C
    IF(IUNIT.EQ.1) GO TO 28
C
54  WRITE(6,30)
30  FORMAT(1X,'HOURLY PRECIPITATION DATA. '//)
    WRITE(6,31)
31  FORMAT(1X,'VALUES ARE IN MILLIMETERS.'////)
    WRITE(6,32)
32  FORMAT(67X,'HOUR ENDING'//)
    WRITE(6,33) (IT(I),I=1,24)
33  FORMAT(11X,24(3X,I2)/)
C
44  CONTINUE
    I7=0
    DO 34 I=1,13
    I4=KYR(I)
    I5=KMO(I)
    I6=KNUMDY(I)
    IF(I.EQ.1) GO TO 35
    I7=I7+KNUMDY(I-1)
35  CONTINUE
    DO 34 J=1,16
    I8=(J-1+I7)*24
    IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 45
    WRITE(6,36) I4,I5,J,(X(K+I8),K=1,24)
36  FORMAT (1X,I2,1X,I2,1X,I2,3X,24F5.1)
    GO TO 34
45  WRITE(6,40) I4,I5,J,(X(K+I8),K=1,24)
46  FORMAT (1X,I2,1X,I2,1X,I2,3X,24F5.2)
34  CONTINUE
C
    IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 47
    WRITE(6,6)
    WRITE(6,62)

```

```

62 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS IN SI UNITS.'//)
   GO TO 50
47 WRITE(6,6)
   WRITE(6,63)
63 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS IN ENGLISH UNITS.'//)
50 CONTINUE
   WRITE(6,41) (IP(I),I=1,16)
41 FORMAT(4X,16(12,6X)/)
C
   WRITE(6,42) ((P(I,J),J=1,16),I=1,MC1)
42 FORMAT(16(1X,F7.2))
   WRITE(6,56) MC1
56 FORMAT(///1X,'NUMBER OF RAINSTORMS=',I4)
C
   IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 51
   GO TO 27
C
28 WRITE(6,30)
   WRITE(6,43)
43 FORMAT(1X,'VALUES ARE IN INCHES.'//)
   WRITE(6,32)
   WRITE(6,33) (IT(I),I=1,24)
   I9=1
   GO TO 44
51 IF(IPRS1.EQ.0) GO TO 27
   I9=2
   DO 52 I=1,I2
52 X(I)=X(I)*25.4
   DO 53 I=1,MC1
   P(I,3)=P(I,3)*25.4
   P(I,4)=P(I,4)*25.4
   P(I,5)=P(I,5)*25.4
53 P(I,10)=P(I,10)*25.4
   WRITE(6,6)
   GO TO 54
C
27 IF(IYR.LT.LASTYR) GO TO 55
C
*****
C*****
C
PROGRAM 2 ...
C
C *** INPUT 2-1
READ(5,80) IPR2
80 FORMAT (I1)
C ***
C
IF(IPR2.EQ.0) GO TO 105
C
DO 83 J=1,16
DO 83 I=1,MC3
83 PA1(I,J)=PA(I,J)
C
OPERATION 2-1 ...
C
C *** INPJ 2-2
READ(5,81) IPR21
81 FORMAT(I1)
C ***
C
IF(IPR21.EQ.0) GO TO 90
C

```

```

      I10=1
      I9=0
      K1=MC3
      GO TO 93
C
C      OPERATION 2-2 ...
C
C *** INPUT 2-3
      90 READ(5,82) IPR22
      82 FORMAT(I1)
C ***
C
C      IF(IPR22.EQ.0) GO TO 98
C
C      I10=2
      I9=0
      K1=MC3
      DO 95 J=1,16
      DO 96 I=1,K1
      A(I)=PA(I,J)
      96 B(I)=PA(I,J)
      CALL SORT(A(I),K1,B(I))
      DO 97 I=1,K1
      97 PA(I,J)=A(I)
      95 CONTINUE
      GO TO 93
C
C      OPERATION 2-3 ...
C
C *** INPUT 2-4
      98 READ(5,85) IPR23
      85 FORMAT(I1)
C ***
C
C      IF(IPR23.EQ.0) GO TO 104
C
C      I10=3
      K1=MC3
      I11=0
C
C *** INPUT 2-5
      READ(5,99) NUMT
      99 FORMAT(I2)
C ***
C
      103 I9=0
      IF(I11.GE.NUMT) GO TO 104
C
C *** INPUT 2-6
      READ(5,94) NOPAR
      94 FORMAT(I2)
C ***
C
      DO 100 J=1,16
      DO 101 I=1,K1
      A(I)=PA(I,NOPAR)
      101 B(I)=PA(I,J)
      CALL SORT(A(I),K1,B(I))
      DO 102 I=1,K1
      102 PA(I,J)=B(I)
      100 CONTINUE
      I11=I11+1
      GO TO 93
C
C      OPERATION 2-4 ...

```



```

C
C *** INPUT 2-7
104 READ(5,126) IPR24
126 FORMAT(I1)
C ***
C
C IF(IPR24.EQ.0) GO TO 105
C
C I10=4
I11=0
C
C *** INPJT 2-8
READ(5,127) NUMT
127 FORMAT(I2)
C ***
C
106 I9=0
IF(I11.GE.NUMT) GO TO 105
C
C *** INPJT 2-9
READ(5,107) NOPAR,XLL,XUL
107 FORMAT(I2,8X,2F10.0)
C ***
C
K1=0
DO 108 I=1,MC3
COUNT=0.
IF(PA(I,NOPAR).GE.XLL.AND.PA(I,NOPAR).LT.XUL) COUNT=1.
IF(COUNT.EQ.0.) GO TO 108
K1=K1+1
DO 109 J=1,16
109 PA(K1,J)=PA(I,J)
108 CONTINUE
DO 121 J=1,16
IF(J.EQ.NOPAR) GO TO 121
DO 122 I=1,K1
A(I)=PA(I,NOPAR)
122 B(I)=PA(I,J)
CALL SORT(A(1),K1,B(1))
DO 123 I=1,K1
123 PA(I,J)=B(I)
121 CONTINUE
DO 129 I=1,K1
129 PA(I,NOPAR)=A(I)
I11=I11+1
C
93 IF(IUNIT.EQ.1) I9=1
91 WRITE(6,6)
GO TO(110,111,112,113),I10
110 WRITE(6,114)
114 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS.'////)
GO TO 120
111 WRITE(6,115)
115 FORMAT(1X,'VALUES OF RAINSTORM PARAMETERS SORTED IN ASCENDING ORDE
1R.'////)
GO TO 120
112 WRITE(6,116) NOPAR
116 FORMAT(1X,'VALUES OF RAINSTORM PARAMETER ',I2,' SORTED IN ASCENDIN
1G ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REARRANGED SI
2MULTANEOUSLY.'////)
GO TO 120
113 IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 118
IF(IUNIT.EQ.2) GO TO 125
IF(NOPAR.NE.3.AND.NOPAR.NE.4.AND.NOPAR.NE.5.AND.NOPAR.NE.10) GO TO
1125

```

```

      XLL=XLL*25.4
      XUL=XUL*25.4
125 WRITE(6,117) NOPAR,XLL,XUL
117 FORMAT(1X,'VALUES OF RAINSTORM PARAMETER ',I2,' GREATER THAN OR EQ
      IUAL TO ',F7.2,' AND LESS THAN ',F7.2,' (IN SI UNITS)'/1X,'SORTED I
      2N ASCENDING ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REA
      3RRANGED SIMULTANEOUSLY.'////)
      GO TO 120
118 WRITE(6,119) NOPAR,XLL,XUL
119 FORMAT(1X,'VALUES OF RAINSTORM PARAMETER ',I2,' GREATER THAN OR EQ
      IUAL TO ',F7.2,' AND LESS THAN ',F7.2,' (IN EN UNITS)'/1X,'SORTED I
      2N ASCENDING ORDER AND CORRESPONDING VALUES OF OTHER PARAMETERS REA
      3RRANGED SIMULTANEOUSLY.'////)
120 CONTINUE
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 87
      WRITE(6,130)
130 FORMAT(1X,'( SI UNITS )'////)
      GO TO 88
      87 WRITE(6,131)
131 FORMAT(1X,'( ENGLISH UNITS )'////)
      88 CONTINUE
      WRITE(6,41) (IP(I),I=1,16)
      WRITE(6,42) ((PA1(I,J),J=1,16),I=1,K1)
      WRITE(6,56) K1
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 89
      84 GO TO (90,98,103,106),I10
      89 IF(IPRS1.EQ.0) GO TO 84
      I9=2
      DO 92 I=1,K1
      PA1(I,3)=PA1(I,3)*25.4
      PA1(I,4)=PA1(I,4)*25.4
      PA1(I,5)=PA1(I,5)*25.4
      92 PA1(I,10)=PA1(I,10)*25.4
      GO TO 91
C
C 105 CONTINUE
E
C *****
C *****
C
C PROGRAM 3 ...
C
C *** INPUT 3-1
C READ(5,150) IPR3
150 FORMAT(I1)
C ***
C
C IF(IPR3.EQ.0) GO TO 151
C
C DO 156 I=1,IDIMPA
156 A(I)=0.0
C
C OPERATION 3-1 ...
C
C *** INPUT 3-2
C READ(5,152) IPR31
152 FORMAT(I1)
C ***
C
C IF(IPR31.EQ.0) GO TO 153
C
C K1=MC3
C I10=1
C I11=0
C

```

```

C *** INPUT 3-3
  READ(5,154) NUMT
154 FORMAT(I2)
C ***
C
  183 I9=0
    IF(I11.GE.NUMT) GO TO 153
C
C *** INPUT 3-4
  READ(5,155) NOPAR,U80(1),U80(3),U80(2)
155 FORMAT(I2,8X,3F10.0)
C ***
C
  DO 157 I=1,MC3
157 A(I)=1.0
    CALL TAB1(PA,A,NOPAR,U80,FREQ,PCT,STATS,IDIMPA,16)
    I11=I11+1
    GO TO 177
C
C
  OPERATION 3-2 ...
C
C *** INPUT 3-5
153 READ(5,184) IPR32
184 FORMAT(I1)
C ***
C
  IF(IPR32.EQ.0) GO TO 151
C
  I10=2
  I11=0
C
C *** INPUT 3-6
  READ(5,185) NUMT
185 FORMAT(I2)
C ***
C
  186 I9=0
    IF(I11.GE.NUMT) GO TO 151
C
C *** INPUT 3-7
  READ(5,187) NOPAR8,NOPAR,XLL,XUL,U80(1),U80(3),U80(2)
187 FORMAT(2I2,6X,5F10.0)
C ***
C
  K1=0
  DO 188 I=1,MC3
    IF(PA(I,NOPAR8).GE.XLL.AND.PA(I,NOPAR8).LT.XUL) A(I)=1.0
    IF(A(I).EQ.0.) GO TO 188
    K1=K1+1
188 CONTINUE
    CALL TAB1(PA,A,NOPAR,U80,FREQ,PCT,STATS,IDIMPA,16)
    I11=I11+1
C
  177 XINTSZ=(U80(3)-U80(1))/(U80(2)-2.)
    IUBQ2=U80(2)
    SUM=0.
    DO 158 I=1,IUBQ2
      PCT(I)=0.01*PCT(I)
      DEN(I)=PCT(I)/XINTSZ
      SUM=SUM+PCT(I)
158 COEN(I)=SUM
C
  160 IF(IUNIT.EQ.1) I9=1
  161 WRITE(6,6)
    IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 169

```

```

      WRITE(6,170)
170 FORMAT(61X,'( SI UNITS )'//)
      GO TO 171
169 WRITE(6,172)
172 FORMAT(58X,'( ENGLISH UNITS )'//)
171 WRITE(6,159) NOPAR
159 FORMAT(1X,'FREQUENCY ANALYSIS FOR RAINSTORM PARAMETER ',I2/)
      IF(I10.EQ.1) GO TO 162
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 164
      IF(IUNIT.EQ.2) GO TO 164
      IF(NOPAR3.NE.3.AND.NOPAR8.NE.4.AND.NOPAR8.NE.5.AND.NOPAR8.NE.10)
160 GO TO 164
      XLL=XLL*25.4
      XUL=XUL*25.4
164 WRITE(6,165) NOPAR8,XLL,XUL
165 FORMAT(1X,'ONLY VALUES CORRESPONDING TO VALUES OF RAINSTORM PARAME
      TER ',I2,' GREATER THAN OR EQUAL TO ',F7.2,' AND LESS THAN ',F7.2,
      2' CONSIDERED.'//)
162 CONTINUE
      WRITE(6,167) UBO(1),UBO(3),UBO(2),XINTSZ
167 FORMAT(//1X,'LOWER LIMIT=',F7.2,5X,'UPPER LIMIT=',F7.2,5X,'NUMBER
      OF CLASS INTERVALS=',F4.0,5X,'INTERVAL SIZE=',F7.2//)
      WRITE(6,168) STATS(4),STATS(5),STATS(2),STATS(3)
168 FORMAT(1X,'MINIMUM=',F7.2,5X,'MAXIMUM=',F7.2,5X,'MEAN=',F7.2,5X,
      1'STANDARD DEVIATION=',F7.2//)
      DO 173 I=2,IUBC2
173 XX(I)=UBO(1)+(I-2)*XINTSZ
      XX(1)=100000.
      XX(IUBC2+1)=100000.
      WRITE(6,175)
175 FORMAT(7X,'INTERVAL',11X,'FREQ',4X,'REL FREQ',4X,'PROB DEN',7X,'N)
      1N-EXCE PROB'//)
      WRITE(6,176) (XX(I),XX(I+1),FREQ(I),PCT(I),DEN(I),CDEN(I),I=1,IUBC
      12)
176 FORMAT(1X,F7.2,3X,F7.2,5X,F7.1,5X,F6.4,5X,F8.4,10X,F6.4)
      WRITE(6,56) K1
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 178
179 GO TO 180
178 IF(IPRS1.EQ.0) GO TO 179
      I9=2
      IF(NOPAR.NE.3.AND.NOPAR.NE.4.AND.NOPAR.NE.5.AND.NOPAR.NE.10) GO TO
1161
      UBO(1)=UBO(1)*25.4
      UBO(3)=UBO(3)*25.4
      XINTSZ=XINTSZ*25.4
      STATS(2)=STATS(2)*25.4
      STATS(3)=STATS(3)*25.4
      STATS(4)=STATS(4)*25.4
      STATS(5)=STATS(5)*25.4
      DO 181 I=1,IUBC2
181 DEN(I)=PCT(I)/XINTSZ
      GO TO 161
180 DO 182 I=1,IDIMPA
182 A(I)=0.0
      GO TO (183,186),I10
C
C 151 CONTINUE
C
C *****
C *****
C
C      PROGRAM 4 ...
C
C *** INPJ 4-1
      READ(5,200) IPR4
200 FORMAT(I1)

```

```

C ***
      IF(IPR4.EQ.0) GO TO 201
C
      DO 202 I=1,IDIMPA
202  A(I)=0.0
      I11=0
C
C *** INPUT 4-2
      READ(5,203) NUMT
203  FORMAT(I2)
C ***
C
204  I9=0
      IF(I11.GE.NUMT) GO TO 201
C
C *** INPUT 4-3
      READ(5,205) NOV(1),UBO2(1,1),UBO2(3,1)
      READ(5,205) NOV(2),UBO2(1,2),UBO2(3,2)
205  FORMAT(I2,8X,2F10.0)
C ***
C
      UBO2(2,1)=20.
      UBO2(2,2)=20.
C
      DO 206 I=1,MC3
206  A(I)=1.0
      CALL TAB2(PA,A,NOV,UBO2,FREQ1,PCT1,STAT1,STAT2,IDIMPA,16)
      I11=I11+1
      XINSZ(1)=(UBO2(3,1)-UBO2(1,1))/(UBO2(2,1)-2.)
      XINSZ(2)=(UBO2(3,2)-UBO2(1,2))/(UBO2(2,2)-2.)
      SUM=0.
      DO 207 I=1,20
      DO 207 J=1,20
      PCT1(I,J)=0.01*PCT1(I,J)
207  DEN1(I,J)=PCT1(I,J)/(XINSZ(1)*XINSZ(2))
C
208  IF(IUNIT.EQ.1) I9=1
209  WRITE(6,6)
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 210
      WRITE(6,236)
236  FORMAT(61X,'( SI UNITS )'//)
      GO TO 211
210  WRITE(6,237)
237  FORMAT(58X,'( ENGLISH UNITS )'//)
211  WRITE(6,212) NOV(1),NOV(2)
212  FORMAT(1X,'TWO-WAY FREQUENCY ANALYSIS FOR RAINSTORM PARAMETERS ',
    1I2,' AND ',I2//)
      DO 213 I=1,2
      WRITE(6,214) NOV(I),UBO2(1,I),UBO2(3,I),UBO2(2,I),XINSZ(I)
214  FORMAT(1X,'RAINSTORM PARAMETER=',I2/1X,'LOWER LIMIT=',F7.2,5X,'UPP
    1ER LIMIT=',F7.2,5X,'NUMBER OF CLASS INTERVALS=',F4.0,5X,'INTERVAL
    2SIZE=',F7.2//)
213  CONTINUE
      DO 216 I=1,21
      IF(I.EQ.1.OR.I.EQ.21) GO TO 217
      XX1(I)=UBO2(1,1)+(I-2)*XINSZ(1)
      GO TO 216
217  XX1(I)=100000.
216  CONTINUE
      DO 218 I=1,21
      IF(I.EQ.1.OR.I.EQ.21) GO TO 219
      XX2(I)=UBO2(1,2)+(I-2)*XINSZ(2)
      GO TO 218
219  XX2(I)=100000.
218  CONTINUE
      DO 215 K=1,4

```



```

      L=(K-1)*5
      WRITE(6,220)
220  FORMAT(///64X,'FREQUENCIES'//)
      WRITE(6,221) NOV(2)
221  FORMAT(5X,'RAINSTORM',45X,'RAINSTORM PARAMETER ',I2/)
      WRITE(6,222) NOV(1), (XX2(I+L),XX2(I+1+L),I=1,5)
222  FORMAT(4X,'PARAMETER ',I2,7X,5(F7.2,2X,F7.2,3X)/)
      WRITE(6,223) (XX1(I),XX1(I+1),FREQ1(I,1+L),FREQ1(I,2+L),FREQ1(I,3+
1L),FREQ1(I,4+L),FREQ1(I,5+L),I=1,20)
223  FORMAT(1X,F7.2,3X,F7.2,9X,F10.4,9X,F10.4,9X,F10.4,9X,F10.4,9X,F10.
14)
215  CONTINUE
      DO 224 K=1,4
      L=(K-1)*5
      WRITE(6,225)
225  FORMAT(///59X,'RELATIVE FREQUENCIES'//)
      WRITE(6,221) NOV(2)
      WRITE(6,222) NOV(1), (XX2(I+L),XX2(I+1+L),I=1,5)
      WRITE(6,223) (XX1(I),XX1(I+1), PCT1(I,1+L), PCT1(I,2+L), PCT1(I,3+
1L), PCT1(I,4+L), PCT1(I,5+L),I=1,20)
224  CONTINUE
      DO 226 K=1,4
      L=(K-1)*5
      WRITE(6,227)
227  FORMAT(///59X,'PROBABILITY DENSITIES'//)
      WRITE(6,221) NOV(2)
      WRITE(6,222) NOV(1), (XX2(I+L),XX2(I+1+L),I=1,5)
      WRITE(6,223) (XX1(I),XX1(I+1), DEN1(I,1+L), DEN1(I,2+L), DEN1(I,3+
1L), DEN1(I,4+L), DEN1(I,5+L),I=1,20)
226  CONTINUE
      WRITE(6,56) MC3
      IF(IUNIT.EQ.1.AND.I9.EQ.1) GO TO 228
229  GO TO 230
228  IF(IPRSI.EQ.0) GO TO 229
      I9=2
      IF(NOV(1).NE.3.AND.NOJ(1).NE.4.AND.NOJ(1).NE.5.AND.NOJ(1).NE.10)
1GO TO 231
      UB02(1,1)=UB02(1,1)*25.4
      UB02(3,1)=UB02(3,1)*25.4
      XINSZ(1)=XINSZ(1)*25.4
231  IF(NOV(2).NE.3.AND.NOJ(2).NE.4.AND.NOJ(2).NE.5.AND.NOJ(2).NE.10)
1GO TO 232
      UB02(1,2)=UB02(1,2)*25.4
      UB02(3,2)=UB02(3,2)*25.4
      XINSZ(2)=XINSZ(2)*25.4
232  DO 233 I=1,20
      DO 233 J=1,20
233  DEN1(I,J)=PCT1(I,J)/(XINSZ(1)*XINSZ(2))
      GO TO 209
230  DO 234 I=1,IDIMPA
234  A(I)=0.0
      GO TO 204
C
201  CONTINUE
C
      STOP
      END

```

```

C*****
C
C SUBROUTINE SORT
C
C IDENTIFICATION
C   SORTS A REAL ARRAY A AND REARRANGES SIMULTANEOUSLY
C   THE CORRESPONDING ELEMENTS OF AN ASSOCIATED REAL ARRAY B.
C
C PURPOSE
C   TO SORT JJ ELEMENTS OF A REAL ARRAY A (BEGINNING AT A(I) AS
C   SPECIFIED BY THE USER) IN ASCENDING ORDER. IN ADDITION,
C   THE CORRESPONDING JJ ELEMENTS OF AN ASSOCIATED REAL ARRAY B
C   (BEGINNING AT B(I) AS SPECIFIED BY THE USER) ARE REARRANGED
C   SIMULTANEOUSLY. SORT ALLOWS SORTING UP TO 2**22-1 ELEMENTS.
C
C USAGE
C   CALL SORT(A(I),JJ,B(I))
C
C DESCRIPTION OF PARAMETERS
C   A(I) -ELEMENT OF ARRAY A AT WHICH SORTING IS TO BEGIN.
C                                     (INPUT-OUTPUT)
C   JJ   -NUMBER OF ELEMENTS OF A,BEGINNING AT A(I),TO BE SORTED.
C                                     (INPUT)
C   B(I) -THE JJ ELEMENTS OF B,BEGINNING AT B(I),ARE REARRANGED
C                                     SIMULTANEOUSLY.
C                                     (INPUT-OUTPUT)
C
C REMARKS
C   IF B IS AN INTEGER ARRAY THEN DELETE STATEMENT 'REAL NT,NTT'
C   AND ADD A NEW STATEMENT 'INTEGER B'.
C*****
C
C SUBROUTINE SORT(A,JJ,B)
C   REAL NT,NTT
C   DIMENSION IU(21),IL(21),A(JJ),B(JJ)
C   M=1
C   II=1
C   I=II
C   J=JJ
5   IF(I.GE.J) GO TO 70
10  K=I
C   IJ=(J+I)/2
C   T=A(IJ)
C   IF(A(I).LE.T) GO TO 20
C   NT=B(IJ)
C   A(IJ)=A(I)
C   B(IJ)=B(I)
C   A(I)=T
C   B(I)=NT
C   T=A(IJ)
20  L=J
C   IF(A(J).GE.T)GO TO 40
C   NT=B(IJ)
C   A(IJ)=A(J)
C   B(IJ)=B(J)
C   A(J)=T
C   B(J)=NT
C   T=A(IJ)
C   IF(A(I).LE.T) GO TO 40
C   NT=B(IJ)
C   A(IJ)=A(I)
C   B(IJ)=B(I)
C   A(I)=T
C   B(I)=NT

```

```

      T=A(IJ)
      GO TO 40
30    NTT=B(L)
      A(L)=A(K)
      B(L)=B(K)
      A(K)=TT
      B(K)=NTT
40    L=L-1
      IF(A(L).GT.T) GO TO 40
      TT=A(L)
50    K=K+1
      IF(A(K).LT.T) GO TO 50
      IF(K.LE.L) GO TO 30
      IF(L-I.LE.J-K) GO TO 60
      IL(M)=I
      IU(M)=L
      I=K
      M=M+1
      GO TO 80
60    IL(M)=K
      IU(M)=J
      J=L
      M=M+1
      GO TO 80
70    M=M-1
      IF(M.EQ.0) RETURN
      I=IL(M)
      J=IU(M)
80    IF(J-I.GE.11) GO TO 10
      IF(I.EQ.II) GO TO 5
      I=I-1
90    I=I+1
      IF(I.EQ.J) GO TO 70
      T=A(I+1)
      IF(A(I).LE.T) GO TO 90
      NT=B(I+1)
      K=I
100   A(K+1)=A(K)
      B(K+1)=B(K)
      K=K+1
      IF(T.LT.A(K)) GO TO 100
      A(K+1)=T
      B(K+1)=NT
      GO TO 90
      END

```

```

C*****
C
C SUBROUTINE TAB1
C
C PURPOSE
C TO TABULATE FOR A GIVEN VARIABLE IN AN OBSERVATION MATRIX,
C THE FREQUENCIES (NUMBER OF OBSERVATIONS) AND PERCENT
C FREQUENCIES OVER GIVEN CLASS INTERVALS. IN ADDITION, FOR THE
C SAME VARIABLE, TOTAL, MEAN, STANDARD DEVIATION, MINIMUM, AND
C MAXIMUM ARE CALCULATED.
C
C USAGE
C CALL TAB1(A,S,NOVAR,UBO,FREQ,PCT,STATS,NO,NV)
C
C DESCRIPTION OF PARAMETERS
C A - INPUT MATRIX OF OBSERVATIONS, NO BY NV
C S - INPUT VECTOR SPECIFYING OBSERVATIONS TO BE CONSIDERED.
C ONLY THOSE OBSERVATIONS WITH A CORRESPONDING NON-ZERO
C S(I) ARE CONSIDERED. VECTOR LENGTH IS NO
C NOVAR - THE VARIABLE TO BE TABULATED.
C UBO - INPUT VECTOR OF LENGTH 3
C UBO(1)= LOWER LIMIT OF THE FIRST CLASS INTERVAL.
C UBO(2)= NUMBER OF CLASS INTERVALS.
C UBO(3)= UPPER LIMIT OF THE LAST CLASS INTERVAL.
C IF UBO(1)=UBO(3), THE PROGRAM USES THE MINIMUM AND
C THE MAXIMUM VALUES OF THE VARIABLE AS UBO(1), AND
C UBO(3), RESPECTIVELY.
C UBO(2) MUST INCLUDE TWO CLASS INTERVALS FOR THE VALUES
C UNDER AND ABOVE LIMITS.
C FREQ - OUTPUT VECTOR OF FREQUENCIES. VECTOR LENGTH IS UBO(2)
C PCT - OUTPUT VECTOR OF PERCENT FREQUENCIES. VECTOR LENGTH
C IS UBO(2)
C STATS - OUTPUT VECTOR OF SUMMARY STATISTICS. VECTOR LENGTH IS 5
C STATS(1)= TOTAL
C STATS(2)= MEAN
C STATS(3)= STANDARD DEVIATION
C STATS(4)= MINIMUM
C STATS(5)= MAXIMUM
C NO - NUMBER OF OBSERVATIONS.
C NV - NUMBER OF VARIABLES.
C
C REMARKS
C INTERVAL SIZE IS COMPUTED AS FOLLOWS
C (UBO(3)-UBO(1))/(UBO(2)-2.)
C A COUNT IS CLASSIFIED INTO A PARTICULAR INTERVAL IF THE VALUE
C IS .GE. THE LOWER LIMIT OF THAT INTERVAL BUT .LT. THE UPPER
C LIMIT OF THE SAME INTERVAL.
C THE DIVISOR FOR STANDARD DEVIATION IS ONE LESS THAN THE NUMBER
C OF OBSERVATIONS USED.
C IF S IS A NULL VECTOR, THEN TOTAL, MEAN, AND STANDARD
C DEVIATION = 0, MIN=1.E75 AND MAX=-1.E75
C SUBROUTINE TAB1 IS IN IBM SYSTEM/360 SCIENTIFIC SUBROUTINE
C PACKAGE VERSION III.
C*****
C
C SUBROUTINE TAB1(A,S,NOVAR,UBO,FREQ,PCT,STATS,NO,NV)
C DIMENSION A(1),S(1),UBO(1),FREQ(1),PCT(1),STATS(1)
C DIMENSION WBO(3)
C DO 5 I=1,3
C 5 WBO(I)=UBO(I)
C
C CALCULATE MIN AND MAX
C
C VMIN=1.0E75

```

```

VMAX=-1.0E75
IJ=NO*(NOVAR-1)
DO 30 J=1,NO
  IJ=IJ+1
  IF(S(J)) 10,30,10
10 IF(A(IJ)-VMIN) 15,20,20
15 VMIN=A(IJ)
20 IF(A(IJ)-VMAX) 30,30,25
25 VMAX=A(IJ)
30 CONTINUE
  STATS(4)=VMIN
  STATS(5)=VMAX
C
C      DETERMINE LIMITS
C
  IF(UBO(1)-UBO(3)) 40,35,40
35 UBO(1)=VMIN
  UBO(3)=VMAX
40 INN=UBO(2)
C
C      CLEAR OUTPUT AREAS
C
  DO 45 I=1,INN
    FREQ(I)=0.0
45 PCT(I)=0.0
    DO 50 I=1,3
      STATS(I)=0.0
C
C      CALCULATE INTERVAL SIZE
C
  SINT=ABS((UBO(3)-UBO(1))/(UBO(2)-2.0))
C
C      TEST SUBSET VECTOR
C
  SCNT=0.0
  IJ=NO*(NOVAR-1)
  DO 75 J=1,NO
    IJ=IJ+1
    IF(S(J)) 55,75,55
55 SCNT=SCNT+1.0
C
C      DEVELOP TOTAL AND FREQUENCIES
C
  STATS(1)=STATS(1)+A(IJ)
  STATS(3)=STATS(3)+A(IJ)*A(IJ)
  TEMP=UBO(1)-SINT
  INTX=INN-1
  DO 60 I=1,INTX
    TEMP=TEMP+SINT
    IF(A(IJ)-TEMP) 70,60,60
60 CONTINUE
    IF(A(IJ)-TEMP) 75,65,65
65 FREQ(INN)=FREQ(INN)+1.0
    GO TO 75
70 FREQ(I)=FREQ(I)+1.0
75 CONTINUE
  IF (SCNT)79,105,79
C
C      CALCULATE PERCENT FREQUENCIES
C
79 DO 80 I=1,INN
80 PCT(I)=FREQ(I)*100.0/SCNT
C
C      CALCULATE MEAN AND STANDARD DEVIATION
C

```



```

      IF(SCNT-1.0) 85,85,90
85  STATS(2)=STATS(1)
    STATS(3)=0.0
    GO TO 95
90  STATS(2)=STATS(1)/SCNT
    STATS(3)=SQRT(ABS((STATS(3)-STATS(1)*STATS(1)/SCNT)/(SCNT-1.0)))
95  DO 100 I=1,3
100  UBO(I)=WBO(I)
105  RETJRN
    END

```

```

C*****
C
C      SUBROUTINE TAB2
C
C      PURPOSE
C      1) PERFORM A TWO-WAY CLASSIFICATION FOR TWO GIVEN VARIABLES IN
C      A. OBSERVATION MATRIX, OF FREQUENCIES (NUMBER OF OBSERVATIONS),
C      PERCENT FREQUENCIES, AND SOME STATISTICS OVER GIVEN CLASS
C      INTERVALS.
C
C      USAGE
C      CALL TAB2(A,S,NOV,UBO,FREQ,PCT,STAT1,STAT2,NO,NV)
C
C      DESCRIPTION OF PARAMETERS
C      A - INPUT MATRIX OF OBSERVATIONS, NO BY NV
C      S - INPUT VECTOR SPECIFYING OBSERVATIONS TO BE CONSIDERED.
C          ONLY THOSE OBSERVATIONS WITH A CORRESPONDING NON-ZERO
C          S(I) ARE CONSIDERED. VECTOR LENGTH IS NO
C      NOV - INPUT VECTOR OF LENGTH 2
C           NOV(1) = NUMBER OF THE FIRST VARIABLE
C                TO BE CROSS-TABULATED.
C           NOV(2) = THE NUMBER OF THE SECOND VARIABLE
C                TO BE CROSS-TABULATED.
C      UBO - INPUT MATRIX OF LENGTH 3 BY 2
C           UBO(1,J) = LOWER LIMIT OF THE FIRST CLASS INTERVAL
C                   FOR THE J TH VARIABLE, J=1,2
C           UBO(2,J) = NUMBER OF CLASS INTERVALS FOR THE
C                   J TH VARIABLE, J=1,2
C           UBO(3,J) = UPPER LIMIT OF THE LAST CLASS INTERVAL
C                   FOR THE J TH VARIABLE, J=1,2
C           IF UBO(1,J)=UBO(3,J), THE PROGRAM USES THE MINIMUM AND
C           THE MAXIMUM VALUES OF THE VARIABLE J AS UBO(1,J) AND
C           UBO(3,J), RESPECTIVELY.
C           UBO(2,J) MUST INCLUDE FOR EACH VARIABLE TWO CLASS
C           INTERVALS FOR THE VALUES UNDER AND ABOVE LIMITS.
C      FREQ - OUTPUT MATRIX OF TWO-WAY CLASSIFICATION OF FREQUENCIES.
C            ORDER OF MATRIX IS INT1 BY INT2, WHERE INT1=UBO(2,1)
C            AND INT2=UBO(2,2)
C      PCT - OUTPUT MATRIX OF TWO-WAY CLASSIFICATION OF PERCENT
C            FREQUENCIES. SAME ORDER AS FREQ
C      STAT1 - OUTPUT MATRIX SUMMARIZING TOTALS, MEANS, AND STANDARD
C            DEVIATIONS FOR EACH CLASS INTERVAL OF VARIABLE 1
C            ORDER OF MATRIX IS 3 BY INT1
C      STAT2 - SAME AS STAT1 BUT FOR VARIABLE 2
C            ORDER OF MATRIX IS 3 BY INT2
C      NO - NUMBER OF OBSERVATIONS.
C      NV - NUMBER OF VARIABLES.
C
C      REMARKS
C      INTERVAL SIZE FOR EACH VARIABLE IS COMPUTED AS FOLLOWS
C      (UBO(3,J)-UBO(1,J))/(UBO(2,J)-2.)
C      FOR EACH VARIABLE, A COUNT IS CLASSIFIED INTO A PARTICULAR
C      INTERVAL IF THE VALUE IS .GE. THE LOWER LIMIT OF THAT INTERVAL
C      BUT .LT. THE UPPER LIMIT OF THE SAME INTERVAL.
C      THE DIVISOR FOR STANDARD DEVIATION IS ONE LESS THAN THE NUMBER
C      OF OBSERVATIONS USED.
C      IF S IS A NULL VECTOR, OUTPUT AREAS ARE SET TO ZERO.
C      SUBROUTINE TAB2 IS IN IBM SYSTEM/360 SCIENTIFIC SUBROUTINE
C      PACKAGE VERSION III.
C*****
C
C      SUBROUTINE TAB2(A,S,NOV,UBO,FREQ,PCT,STAT1,STAT2,NO,NV)
C      DIMENSION A(1),S(1),NOV(2),UBO(3,2),FREQ(1),PCT(1),STAT1(1),
C      1STAT2(2),SINT(2)
C      DIMENSION WBO(3,2)

```

```

      DO 5 I=1,3
      DO 5 J=1,2
5     WBO(I,J)=UBO(I,J)
C
C     DETERMINE LIMITS
C
      DO 40 I=1,2
      IF(UBO(I,1)-UBO(3,I)) 40, 10, 40
10    VMIN=1.0E75
      VMAX=-1.0E75
      IJ=NO*(NOV(I)-1)
      DO 35 J=1,NO
      IJ=IJ+1
      IF(S(J)) 15,35,15
15    IF(A(IJ)-VMIN) 20,25,25
20    VMIN=A(IJ)
25    IF(A(IJ)-VMAX) 35,35,30
30    VMAX=A(IJ)
35    CONTINUE
      UBO(1,I)=VMIN
      UBO(3,I)=VMAX
40    CONTINUE
C
C     CALCULATE INTERVAL SIZE
C
45    DO 50 I=1,2
50    SINT(I)=ABS((UBO(3,I)-UBO(1,I))/(UBO(2,I)-2.0))
C
C     CLEAR OUTPUT AREAS
C
      INT1=UBO(2,1)
      INT2=UBO(2,2)
      INTT=INT1*INT2
      DO 55 I=1,INTT
      FREQ(I)=0.0
55    PCT(I)=0.0
      INTY=3*INT1
      DO 60 I=1,INTY
60    STAT1(I)=0.0
      INTZ=3*INT2
      DO 65 I=1,INTZ
65    STAT2(I)=0.0
C
C     TEST SUBSET VECTOR
C
      SCNT=0.0
      INTY=INT1-1
      INTX=INT2-1
      IJ=NO*(NOV(1)-1)
      IJX=NO*(NOV(2)-1)
      DO 95 J=1,NO
      IJ=IJ+1
      IJX=IJX+1
      IF(S(J)) 70,95,70
70    SCNT=SCNT+1.0
C
C     CALCULATE FREQUENCIES
C
      TEMP1=UBO(1,1)-SINT(1)
      DO 75 IY=1,INTY
      TEMP1=TEMP1+SINT(1)
      IF(A(IJ)-TEMP1) 80,75,75
75    CONTINUE
      IY=INT1
80    IYY=3*(IY-1)+1

```

```

      STAT1(IYY)=STAT1(IYY)+A(IJ)
      IYY=IYY+1
      STAT1(IYY)=STAT1(IYY)+1.0
      IYY=IYY+1
      STAT1(IYY)=STAT1(IYY)+A(IJ)*A(IJ)
      TEMP2=UBO(1,2)-SINT(2)
      DO 85 IX=1,INTX
      TEMP2=TEMP2+SINT(2)
      IF(A(IJX)-TEMP2) 90,85,85
85  CONTINUE
      IX=INT2
90  IJF=INT1*(IX-1)+IY
      FREQ(IJF)=FREQ(IJF)+1.0
      IX=3*(IX-1)+1
      STAT2(IX)=STAT2(IX)+A(IJX)
      IX=IX+1
      STAT2(IX)=STAT2(IX)+1.0
      IX=IX+1
      STAT2(IX)=STAT2(IX)+A(IJX)*A(IJX)
95  CONTINUE
      IF (SCNT)98,151,98
C
C      CALCULATE PERCENT FREQUENCIES
C
98  DO 100 I=1,INTT
100 PCT(I)=FREQ(I)*100.0/SCNT
C
C      CALCULATE TOTALS, MEANS, STANDARD DEVIATIONS
C
      IXY=-1
      DO 120 I=1,INT1
      IXY=IXY+3
      ISD=IXY+1
      TEMP1=STAT1(IXY)
      SUM=STAT1(IXY-1)
      IF(TEMP1-1.0) 120,105,110
105  STAT1(ISD)=0.0
      GO TO 115
110  STAT1(ISD)=SQRT(ABS((STAT1(ISD)-SUM*SUM/TEMP1)/(TEMP1-1.0)))
115  STAT1(IXY)=SUM/TEMP1
120  CONTINUE
      IXX=-1
      DO 140 I=1,INT2
      IXX=IXX+3
      ISD=IXX+1
      TEMP2=STAT2(IXX)
      SUM=STAT2(IXX-1)
      IF(TEMP2-1.0) 140,125,130
125  STAT2(ISD)=0.0
      GO TO 135
130  STAT2(ISD)=SQRT(ABS((STAT2(ISD)-SUM*SUM/TEMP2)/(TEMP2-1.0)))
135  STAT2(IXX)=SUM/TEMP2
140  CONTINUE
      DO 150 I=1,3
      DO 150 J=1,2
150  UBO(I,J)=WBO(I,J)
151  RETURN
      END

```

APPENDIX B

DATA PREPARATION AND LISTING OF COMPUTER PROGRAM FOR ILLINOIS SURFACE RUNOFF MODEL

The Illinois Surface Runoff Model is programmed in Fortran IV language for computer solutions as described in Sec. VI-4 and VIII-2(A). The input to the computer program is the drainage basin characteristics and rainfall hyetographs. The output is the subcatchment hydrographs and pollutographs which serve as the input to the Illinois Storm Sewer System Simulation Model and the Illinois Sewer System Water Quality Model, respectively. The storage requirement of the computer program is 400K bytes. The potential program user is urged to read Sec. VI-4 and VIII-2 (A) before proceeding to read the following description on data preparation and program listing.

(a) Data Preparation

Detailed information on the basin characteristics and rainfall is needed for the Illinois Surface Runoff Model and hence a number of data cards are required for the computer program. The data deck consists of the following five sets in the order of presentation. In each of Figs. B-1 to B-4 the values of the first example are identical with those given as example in the Entry in the description of the corresponding sets.

Set 1. Measurement unit — This set consists of one card specifying whether the data is provided in English or SI measurement units. Consistent units should be used for all the data. The computed results will be printed out with the units identical with the input. The format of this card is as follows:

Card	Column	Format	Entry
1-1	1	I1	= 1 for English units = 2 for SI units
1-1	2-80	Blank	

In the card identification the numeral before the dash indicates the set number and the numeral after the dash represents the number of the card following its order in the set.

Set 2. General description of drainage basin — This set consists of one card as follows:

Card	Column	Format	Entry
2-1	1-5	I5	Total number of gutters in the system; e.g., 25
2-1	6-10	I5	Total number of sewer nodes in the system; e.g., 20

Card	Column	Format	Entry
2-1	11-15	I5	Total number of rain-zones within the basin, maximum 5; e.g., 3
2-1	16-20	I5	Frequency of printed output; e.g., 2 if results are to be printed out at every other time interval specified in Col. 21-30
2-1	21-30	F10.0	Time interval of computation in min e.g., for $\Delta t = 1.0$ min, one can enter 0001.00000 or 1.00000000 or 01.0000000 or 000000001. etc.
2-1	31-40	F10.0	Time in min when the execution should stop; e.g., 100. represents the computation terminates at 100 min
2-1	41-50	F10.0	Value of gravitational acceleration in ft/sec^2 or m/sec^2 ; e.g., 32.2 ft/sec^2
2-1	51-60	F10.0	Average daily temperature on the day of rainstorm in degrees of Fahrenheit (for English units) or Celsius (for metric units). Leave blank when no snowmelting is involved
2-1	61-70	F10.0	Kinematic viscosity of water in gutters in ft^2/sec or mm^2/sec ; e.g. 0.00001 ft^2/sec
2-1	71-80	F10.0	The laminar flow resistance constant C in Eq. 48. Unless any other value is desired 24.0 should be punched

The time interval and the stop execution values should be selected such that the latter must be an exact multiple of the former, and there should be no more than 100 time steps of computation. Three examples for data card Sets 1 and 2 are given in Fig. B-1.

Set 3. Hyetographs — This set consists of as many subsets as the number of rain-zones within the basin (maximum 5). If more than one rain-zone is considered, they should be numbered in ascending order starting from 1, e.g., 1, 2, and 3 for three rain-zones. Each subset describes a hyetograph for a rain-zone. The first subset is for the hyetograph for Rain-Zone 1, the second subset for Rain-Zone 2, etc. For each hyetograph there should be no more than 100 pairs of time-rainfall intensity values and the time values should be given in increasing order. Each subset contains as few as 2 cards to as many as 14 cards depending on the number of pairs of time-intensity data of the hyetograph. The cards for each subset are described as follows:

Card	Column	Format	Entry
3-1	1-10	F10.0	Time rainstorm begins in min; e.g., 144. indicates the rainstorm begins at 2:24 a.m. of the date

Example Set	Entry																							
	Col.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	1	1	25	20	3	2	1.0				100.			32.2										24.
	2	1	2																					
2	1	15	14	1	2	2					100.0			9.81						15.0				24.0
	2	1																						
3	1	1																						
	2	18	15	1	1	1	3.0				150.00			32.2						35.00				24.00

Fig. B-1. Examples of data card Sets 1 and 2 for Illinois Surface Runoff Model

Example Set	Entry																							
	Col.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	1	144.	0.	147.	2.5	149.	3.5	155.	4.0	158.	5.0	159.	4.0	160.	3.0	162.	1.5							
	2	163.	1.0	164.	0.0																			
2	1	115.0	0.	115.0	0.	2.0																		
	2	115.	0.	150.	1.0	140.	2.5	155.	5.	160.	7.5	190.	6.0	210.	4.	235.	0.							

Fig. B-2. Examples of data card Set 3 for hyetographs for Illinois Surface Runoff Model

Card	Column	Format	Entry
3-1	11-20	F10.0	Time rainstorm ceases in min; e.g., 164. indicates the rainstorm ceases at 2:44 a.m. of the date
3-1	21-30	F10.0	Total daily rainfall on the day of rainstorm in in. or mm, leave blank if snowmelt is not considered
3-1	31-35	I5	Total number of pairs of time and intensity values used to describe the hyetograph, maximum 100; e.g., 10
3-1	36-80	Blank	
3-2	1-5	F5.0	Time of the first rainfall data in min; e.g. 144. min
3-2	5-10	F5.0	Intensity in in./hr or mm/hr corresponding to the time in Col. 1-5; e.g., 0. in./hr
3-2	11-15	F5.0	Time of the second rainfall data in min; e.g., 147. min
3-2	16-20	F5.0	Intensity in in./hr or mm/hr corresponding to the time in Col. 11-15; e.g., 2.5 in./hr
3-2	21-25	F5.0	Time of rainfall data in min; e.g., 149. min
3-2	26-30	F5.0	Intensity corresponding to the time in Col. 21-25; e.g., 3.5 in./hr
3-2	31-35	F5.0	Time of rainfall in min
3-2	36-40	F5.0	Intensity corresponding to the time in Col. 31-35
3-2	41-45	F5.0	Time in min
3-2	46-50	F5.0	Intensity corresponding to the time in Col. 41-45
3-2	51-55	F5.0	Time in min
3-2	56-60	F5.0	Intensity corresponding to the time in Col. 51-55
3-2	61-65	F5.0	Time in min
3-2	66-70	F5.0	Intensity corresponding to the time in Col. 61-65
3-2	71-75	F5.0	Time in min
3-2	76-80	F5.0	Intensity corresponding to the time in Col. 71-75
3-3			The above data card (Card 3-2) should be repeated until the entire hyetograph is accounted for
.			
.			
(3-14 max)			

Two examples of data card Set 3 are given in Fig. B-2.

Set 4. Gutter, inlet and subcatchment descriptions — This set contains a number of subsets, each corresponding to a gutter considered in the system. Each gutter is given the same number as the inlet at its downstream end and also identified by its upstream inlet or inlets. When there is no inlet at the downstream end of the gutter an imaginary inlet (Type=0.0) should be assigned. The types of gutters and inlets considered in the computer program are each represented by a number as shown in Fig. 24. When an imaginary inlet is introduced, the flow into the inlet is always zero, and the gutter outflow is equal to the carry-over from the imaginary inlet to the following gutter or gutters. For an inlet located in a valley receiving all the water from the contributing gutters, an imaginary gutter (Type=0.0) should be assigned so that the program will call the subroutine STROUT to perform the storage routing. Each subset consists of 4 to 13 cards depending on the number of subcatchment strips of the gutter. The first card of the subset identifies the location and geometry associated with the gutter. The second card contains data on infiltration and initial pollution concentrations. The third card identifies the inlets at the upstream end of the gutter and the corresponding proportion of carry-over water. The fourth and following cards (up to thirteenth maximum) describe the subcatchment characteristics.

Card	Column	Format	Entry
4-1	1-5	I5	The gutter number; e.g., 2
4-1	6-10	I5	Number of grid points for gutter routing = one plus number of reaches the length of the gutter is to be equally divided; e.g., 3
4-1	11-15	I5	The rain-zone number to which the gutter belongs; e.g., 5
4-1	16-20	F5.0	Type of gutter (see Fig. 24); e.g., 1.0
4-1	21-25	F5.0	Length of gutter in ft or m; e.g., 250.0 ft
4-1	26-30	F5.0	Width of gutter in ft or m; e.g. 2.0 ft
4-1	31-35	F5.0	Longitudinal slope of gutter; e.g., 0.01 (=1%)
4-1	36-40	F5.0	Depth of gutter for rectangular gutters in ft or m, leave blank for gutters of other cross-sectional shape
4-1	41-45	F5.0	Angle between the gutter plane and the vertical in radians; e.g., 1.5 rad.
4-1	46-50	F5.0	Type of inlet at the end of the gutter (see Fig. 24); e.g., 1.0
4-1	51-55	F5.0	Width of inlet in ft or m; e.g., 1.0 ft
4-1	56-60	F5.0	Length of inlet in ft or m; e.g., 2.0 ft
4-1	61-65	F5.0	Ratio of total area of openings to total area of inlet; e.g., 0.60

Card	Column	Format	Entry
4-1	66-70	F5.0	Width of street pavement measured from the crown to the gutter in ft or m; e.g., 16.0 ft
4-1	71-75	F5.0	Initial depth along the gutter in ft or m; e.g., 0.0001 ft. The value 0.0001 ft or 0.0001 m should be punched when no other value is desired
4-1	76-80	F5.0	Manning's roughness factor for the gutter; e.g., $n=0.016$
4-2	1-5	F5.0	Initial infiltration capacity of gutter surface in in./hr or mm/hr; e.g., $f_0=0.05$ in./hr
4-2	6-10	F5.0	Final infiltration capacity of gutter surface in in./hr or mm/hr; e.g., $f_c=0.01$ in./hr
4-2	11-15	F5.0	Horton's constant of decay rate of infiltration for gutter surface in hr^{-1} ; e.g., $k=15.0 \text{ hr}^{-1}$
4-2	16-20	F5.0	Initial infiltration capacity for street pavement in in./hr or mm/hr; e.g., $f_0=0.05$ in./hr
4-2	21-25	F5.0	Final infiltration capacity for street pavement in in./hr or mm/hr; e.g., $f_c=0.02$ in./hr
4-2	26-30	F5.0	Horton's constant of decay rate of infiltration for street pavement in hr^{-1} ; e.g., $k=10.0 \text{ hr}^{-1}$
4-2	31-35	F5.0	Initial concentration of the first pollutant associated with gutter flow in ppm or mg/l; e.g., 0.001 ppm
4-2	36-40	F5.0	Initial concentration of the second pollutant associated with gutter flow in ppm or mg/l; e.g., 0.3 ppm
4-2	41-80	Blank	
4-3	1-5	I5	The number of the 1st immediately upstream inlet; e.g., 1
4-3	6-10	I5	The number of the 2nd immediately upstream inlet; e.g., 5
4-3	11-15	I5	The number of the 3rd immediately upstream inlet; e.g., 10
4-3	16-20	I5	The number of the 4th immediately upstream inlet; e.g., 6
4-3	21-25	I5	The number of the 5th immediately upstream inlet; e.g., 12

Card	Column	Format	Entry
4-3	26-30	I5	The number of the 6th immediately upstream inlet
4-3	31-35	F5.0	Proportion of carry-over taken by gutter from 1st upstream inlet; e.g., 1.0
4-3	36-40	F5.0	Proportion of carry-over taken by gutter from 2nd upstream inlet; e.g., 0.5
4-3	41-45	F5.0	Proportion of carry-over taken by gutter from 3rd upstream inlet; e.g., 0.8
4-3	46-50	F5.0	Proportion of carry-over taken by gutter from 4th upstream inlet; e.g., 0.8
4-3	51-55	F5.0	Proportion of carry-over taken by gutter from 5th upstream inlet; e.g., 0.8
4-3	56-60	F5.0	Proportion of carry-over taken by gutter from 6th upstream inlet
4-3	61-80	Blank	
4-4	1-5	F5.0	Length of the 1st (most upstream one along the gutter) subcatchment strip in ft or m; e.g., 80.0 ft
4-4	6-10	F5.0	Average slope of the strip; e.g., 0.01 (=1%)
4-4	11-15	F5.0	Surface roughness in ft or m; e.g., 0.01 ft
4-4	16-20	F5.0	Initial infiltration capacity in in./hr or mm/hr; e.g., 1.0 in./hr
4-4	21-25	F5.0	Final infiltration capacity in in./hr or mm/hr; e.g., 0.3 in./hr
4-4	26-30	F5.0	Horton's constant of decay rate of infiltration in hr^{-1} ; e.g., 5.0 hr^{-1}
4-4	31-35	F5.0	Initial depth along the strip in ft or m; 0.0001 ft or 0.0001 m should be punched if no other value is desired
4-4	36-40	F5.0	Initial concentration of the first pollutant associated with the subcatchment flow in ppm or mg/l; e.g., 0.005 ppm
4-4	41-45	F5.0	Initial concentration of the second pollutant associated with the subcatchment flow in ppm or mg/l; e.g., 0.72 ppm
4-4	46-50	I5	Number of computation grid points along the strip, (minimum is 2); e.g., 4

Card	Column	Format	Entry
4-4	51-80	Blank	
4-5			Repeat as the above data card (4-4) for
.			all the subcatchment strips starting
.			from the most upstream strip and pro-
(4-13 max)			ceeding towards downstream. Maximum
			10 cards (strips) for each subset
			(gutter)

When using an imaginary gutter (Type=0.0), the area of the storage surface in ft^2 or m^2 is punched as the fifth quantity (Col. 21-25) on Card 4-1 and the integer number 2 should be punched in Col. 10. In this case the data cards 4-2 and 4-4 each can be left blank. For Card 4-3, if there are less than six upstream inlets, punch only those existing in successive order and leave the rest of the spaces blank. If there is no upstream inlet at all, this card should still be there with nothing punched on it.

Two examples of data card Set 4 are shown in Fig. B-3.

Set 5. Sewer node description — In this data set one card is needed to describe each sewer node in the system. Since a maximum of 100 sewer nodes is allowed, this set consists of a minimum of one card and a maximum of 100 cards. The information contained in each card is as follows:

Card	Column	Format	Entry
5-1	1-5	I5	Sewer node number; e.g., 5
5-1	6-10	F5.0	Type of sewer node (see Fig. 24), = 1.0 for junctions = 2.0 for upstream nodes with no incoming sewers
5-1	11-15	F5.0	Base flow for the sewer node in cfs or m^3/sec ; e.g., 1.0 cfs
5-1	16-20	F5.0	Concentration of the first pollutant in base flow in ppm or mg/ℓ ; e.g., 0.05 ppm
5-1	21-25	F5.0	Concentration of the second pollutant in base flow in ppm or mg/ℓ ; e.g., 0.1 ppm
5-1	26-30	I5	Number of 1st inlet discharging into the sewer node; e.g., 2
5-1	31-35	I5	Number of 2nd inlet discharging into the sewer node; e.g., 4
5-1	36-40	I5	Number of 3rd inlet discharging into the sewer node; e.g., 9
5-1	41-45	I5	Number of 4th inlet discharging into the sewer node; e.g., 7
			Number of 5th inlet discharging into the sewer node; e.g., 10

Example	Col.	Entry																																
1	2	3	5	1.0	250.0	2.0	.01																											
	.05	.01	15.	.05	.02	10.	.001	.3																										
	1	5	10	6	12		1.0	0.5	0.8	0.8																								
	80.	.01	.01	1.0	0.3	5.0	.0001	.005	.72																									
2	60.	.05	.01	1.5	0.5	4.0	.0001	.005	.001																									
	8	4	2	2.0	150.	2.	.005	0.1	0.	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	
	.05	.02	20.	.05	.02	15.	.005	.002																										
	15																																	
	0.	0.	0.	0.	0.0	0.	0.	0.0	0.																									
	0.	0.	0.	0.0	0.0	0.	0.	0.	0.																									
	50.	.06	.01	1.0	0.3	5.0	.0001	.005	.001																									

Fig. B-3. Examples of data card Set 4 for subcatchments for Illinois Surface Runoff Model

Example		Entry																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
Col.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														

Fig. B-4. Examples of data card Set 5 for sewer nodes for Illinois Surface Runoff Model

Card	Column	Format	Entry
5-1	46-50	I5	Number of 5th inlet discharging into the sewer node; e.g., 10
5-1	51-55	I5	Number of 6th inlet discharging into the sewer node
5-1	56-60	I5	Number of 7th inlet discharging into the sewer node
5-1	61-65	I5	Number of 8th inlet discharging into the sewer node
5-1	66-70	I5	Number of 9th inlet discharging into the sewer node
5-1	71-75	I5	Number of 10th inlet discharging into the sewer node
5-1	76-80	Blank	
5-2			Repeat the above data card (5-1) for each sewer node in the system until all the sewer nodes are accounted for.
.			
.			
(5-100 max)			

In Col. 26-75, if there are less than 10 inlets contributing to the sewer node, fill in only those existing in successive order and leave the excessive spaces blank. An example of data cards for Set 5 is given in Fig. B-4.

A schematic drawing showing the order of input data card decks for the Illinois Surface Runoff Model is given in Fig. B-5. As a more Comprehensive illustration on data preparation, the data representation for a conceptual simple drainage basin shown in Fig. B-6 is given in Fig. B-7 as an example.

(b) Printed Output

Inflow hydrographs for each of the sewer nodes together with the associated pollutant concentrations are printed as the output of the computer program of the Illinois Surface Runoff Model. A sample print-out for sewer node No. 33 is shown in Fig. B-8. When the user is concerned merely with the quantity of water, the results of the quality computations should be disregarded. It should be noted that the time-discharge values are obtained also on cards to be used as input to the ISS Model.

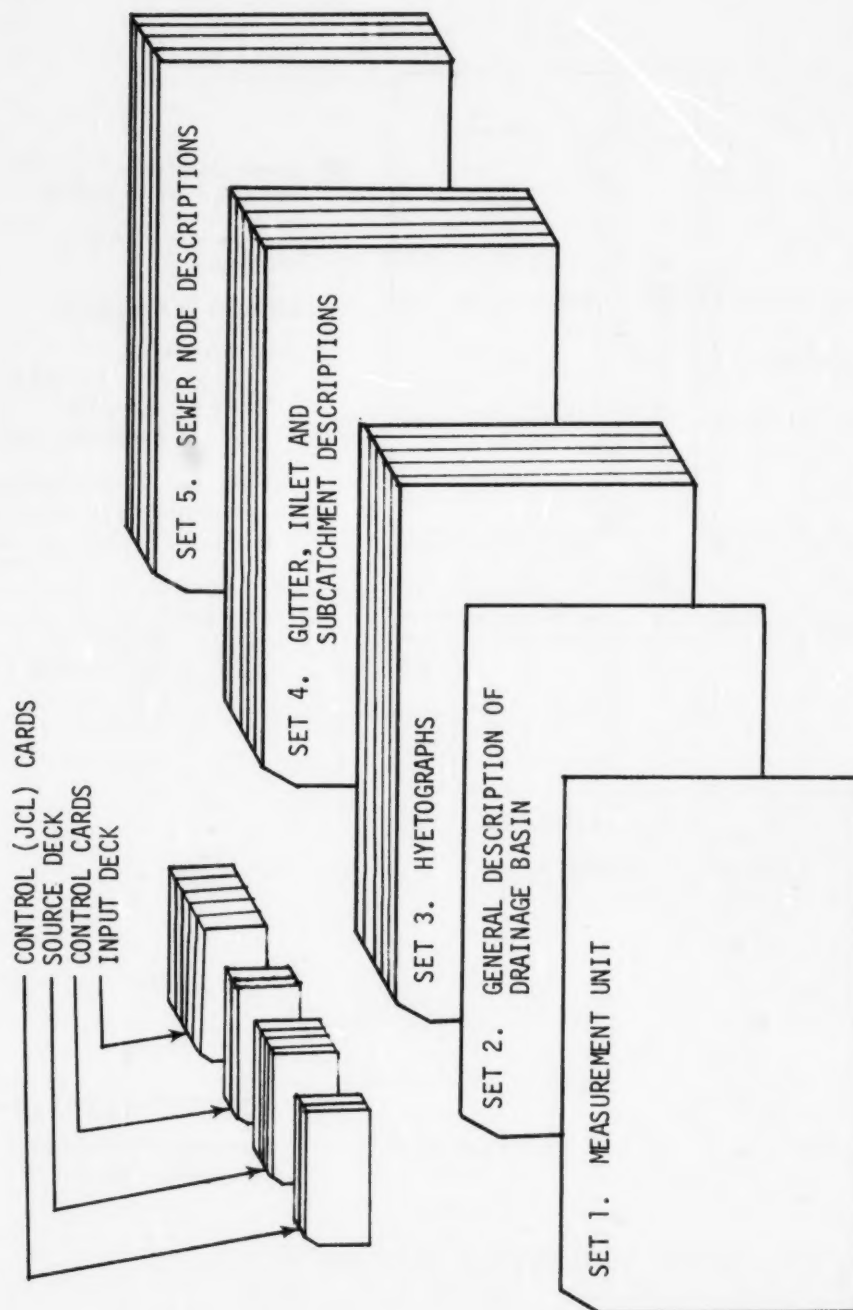


Fig. B-5. Arrangement of input card decks for Illinois Surface Runoff Model



Note: b indicates blank card

199

33 FLOW INTO SEWER JUNCTION=

TIME (SEC)	DISCHARGE (CFS)	1ST POLLUTANT CONCENTRATION	2ND POLLUTANT CONCENTRATION
0.00	1.00	0.36	0.85
60.00	1.10	0.52	0.93
120.00	1.47	0.89	1.10
180.00	1.89	0.93	1.05
240.00	2.20	0.83	0.93
300.00	1.94	1.08	1.17
360.00	1.70	1.40	1.46
420.00	1.54	1.72	1.75

Fig. B-8. Sample output from computer program of Illinois Surface Runoff Model

(c) Program Listing

```

C
C      MAIN PROGRAM FOR ILLINOIS SURFACE RUNOFF MODEL
C
C      MAIN PROGRAM PERFORMS THE ROUTING COMPUTATIONS FOR QUANTITY AND
C      QUALITY OF GUTTER FLOW
C
COMMON/Z1/TEMP,IRYN,RYNST,RYNEND,DAYRYN,TH,RAIN
COMMON/Z2/FFIN,FINH,CINF,QRAIN,QRST
COMMON/Z3/XNU,C1,FK,SO,ULL,NUV,YUIN,DT,QUVERL
COMMON/Z4/QCACH,TBA,GRBTYP,W,GRBL,GU,T,VP,YP,UP,FL0LAT,OPR
COMMON/Z5/SG,RAFCUF,YING,COTTH,SINTH,COSTH,TANTH,YUPST,QUPST
COMMON/Z20/CUNIN1,CUNIN2,COVER1,COVER2
COMMON/Z6/K1,K2,K3,K4,K5,OR1,OR2,OR3,OR4,OR5,OR6,K5
COMMON/Z7/I,GTTRYP,B,TIME,WUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
* C1C1,C1C2
COMMON/Z8/G,L1,LYH
COMMON/Z9/JTOTAL,INLET,LIMIT,TYHEND,DTH,NSW,SHRTYP,BYSFLO
COMMON/Z10/L2,L3,MDTH,NGTR
COMMON/Z11/GL
COMMON/Z12/CJ1,CJ2
DIMENSION CJ1(100),CJ2(100)
DIMENSION RYNST(5),RYNEND(5),TH(5,100),RAIN(5,100),DAYRYN(5)
DIMENSION NUKTA(5)
DIMENSION QCACH(100,100),TBA(100),GRBTYP(100),W(100),GRBL(100)
DIMENSION GTTRYP(100),B(100),TIME(100),QUP(100,100),OPR(100),NGTR(
* 100)
DIMENSION SG(100),RAFCUF(100),YING(100)
DIMENSION K1(100),K2(100),K3(100),K4(100),K5(100),K6(100)
DIMENSION OR1(100),OR2(100),OR3(100),OR4(100),OR5(100),OR6(100)
DIMENSION GU(11),VU(11),YU(11),YU(11)
DIMENSION NU(100,10),FFINAL(100,10),FINSHL(100,10),CINFLE(100,10)
DIMENSION DK(100,10),DS(100,10),UL(100,10),YIND(100,10)
DIMENSION TRNZN(100),N(100),PL(100),FFINP(100),FINSP(100)
DIMENSION CGTR1(100),CGTR2(100),CU1(100,10),CU2(100,10)
DIMENSION CCC1(100,100),CCC2(100,100)
DIMENSION DUMMY(100),GL(100),FFING(100),FINSG(100),CINFG(100)
DIMENSION INLET(100,10),NSW(100),SHRTYP(100),BYSFLO(100)
DIMENSION W(100),CINF(100)
INTEGER UNIT

C
C      FOLLOWING ARE THE STATEMENT FUNCTION DESCRIBING CROSS-SECTIONAL
C      PROPERTIES AND THE FRICTION SLOPE EVALUATION FOR TWO TYPES OF GUTTER
C
A1(E)=0.5*E**2/COTTH
T1(E)=E/COTTH
R1(E)=0.5*E*SINTH/(1.+COSTH)
A1P(E)=E/COTTH
T1P(E)=1./COTTH
R1P(E)=0.5*SINTH/(1.0+COSTH)
A1PP(E)=1./COTTH
R1PP(E)=0.
R1PPP(E)=0.
A2(E)=U*E-0.5*COTTH*U**2
T2(E)=U
R2(E)=(U*E-0.5*COTTH*U**2)/(E+(U/SINTH))
A2P(E)=U
R2P(E)=(U**2/SINTH+0.5*COTTH*U**2)/(E+(U/SINTH))**2
T2P(E)=0.0
A2PP(E)=0.
R2PPP(E)=(U**2/SINTH+0.5*COTTH*U**2)*(-2./(E+U/SINTH))**3
R2PPP(E)=6.*(U**2/SINTH+COTTH*U**2/2.)/(E+(U/SINTH))**4
A3(E)=U*E

```

```

T3(E)=U
R3(E)=U*E/(U+2.*E)
A3P(E)=U
T3PP(E)=0.0
R3P(E)=U**2/(U+2.*E)**2
A3PP(E)=0.
R3PPP(E)=U**2*(-4./(U+2.*E)**3)
R3PP(E)=24.*U**2/(U+2.*E)**4
A4(E)=U*E
T4(E)=U
R4(E)=U*E/(U+HHHH+E)
A4P(E)=U
R4P(E)=(U**2+U*HHHH)/(U+HHHH+E)**2
T4PP(E)=0.0
A4PP(E)=0.0
R4PP(E)=0.0
R4PPP(E)=6.*(U**2+U*HHHH)/(U+HHHH+E)**4
G1(E)=CDF*(.67*A1(E)*R1P(E)/R1(E)**.33+A1P(E)*R1(F)**.67)
G2(E)=CDF*(.67*A2(E)*R2P(E)/R2(E)**.33+A2P(E)*R2(F)**.67)
G3(E)=CDF*(.67*A3(E)*R3P(E)/R3(E)**.33+A3P(E)*R3(F)**.67)
G4(E)=CDF*(.67*A4(E)*R4P(E)/R4(E)**.33+A4P(E)*R4(F)**.67)
G1P(E)=CDF*(A1PP(E)*R1(E)**.67+1.33*A1P(E)*R1P(E)/R1(E)**.33-.22*A
*1(E)*R1P(E)**2/R1(E)**1.33+.67*A1(E)*R1PP(E)/R1(E)**.33)
G2P(E)=CDF*(A2PP(E)*R2(E)**.67+1.33*A2P(E)*R2P(E)/R2(E)**.33-.22*A
*2(E)*R2P(E)**2/R2(E)**1.33+.67*A2(E)*R2PP(E)/R2(E)**.33)
G3P(E)=CDF*(A3PP(E)*R3(E)**.67+1.33*A3P(E)*R3P(E)/R3(E)**.33-.22*A
*3(E)*R3P(E)**2/R3(E)**1.33+.67*A3(E)*R3PP(E)/R3(E)**.33)
G4P(E)=CDF*(A4PP(E)*R4(E)**.67+1.33*A4P(E)*R4P(E)/R4(E)**.33-.22*A
*4(E)*R4P(E)**2/R4(E)**1.33+.67*A4(E)*R4PP(E)/R4(E)**.33)
G1PP(E)=CDF*(2.*A1P(E)*R1P(E)/R1(E)**.33-.67*A1P(E)*R1P(E)**2/R1(
*E)**1.33+2.*A1P(E)*R1PP(E)/R1(E)**.33-.67*A1(E)*R1P(F)*R1PP(E)/R1(
*E)**1.33+.296*A1(E)*R1P(E)**3/R1(E)**2.34+.67*A1(F)*R1PPP(E)/R1(F)
***.33)
G2PP(E)=CDF*(2.*R2P(E)*A2PP(E)/R2(E)**.33-.67*A2P(E)*R2P(E)**2/R2(
*E)**1.33+2.*A2P(E)*R2PP(E)/R2(E)**.33-.67*A2(E)*R2P(F)*R2PP(E)/R2(
*E)**1.33+.296*A2(E)*R2P(E)**3/R2(E)**2.34+.67*A2(F)*R2PPP(E)/R2(F)
***.33)
G3PP(E)=CDF*(2.*R3P(E)*A3PP(E)/R3(E)**.33-.67*A3P(E)*R3P(E)**2/R3(
*E)**1.33+2.*A3P(E)*R3PP(E)/R3(E)**.33-.67*A3(E)*R3P(F)*R3PP(E)/R3(
*E)**1.33+.296*A3(E)*R3P(E)**3/R3(E)**2.34+.67*A3(F)*R3PPP(E)/R3(E)
***.33)
G4PP(E)=CDF*(2.*R4P(E)*A4PP(E)/R4(E)**.33-.67*A4P(E)*R4P(E)**2/R4(
*E)**1.33+2.*A4P(E)*R4PP(E)/R4(E)**.33-.67*A4(E)*R4P(F)*R4PP(E)/R4(
*E)**1.33+.296*A4(E)*R4P(E)**3/R4(E)**2.34+.67*A4(F)*R4PPP(E)/R4(F)
***.33)
Q1(E)=CDF*A1(E)*R1(E)**0.67
Q2(E)=CDF*A2(E)*R2(E)**0.67
Q3(E)=CDF*A3(E)*R3(E)**0.67
Q4(E)=CDF*A4(E)*R4(E)**0.67
F1(E)=F=(HET=.25*GAM*T1(E)/DT+.5*G1(E)*YD/DX)/( .25*T1(E)/DT+.5*G1(
*E)/DX+ALFA)
F2(E)=F=(HET=.25*GAM*T2(E)/DT+.5*G2(E)*YU/DX)/( .25*T2(E)/DT+.5*G2(
*E)/DX+ALFA)
F3(E)=F=(HET=.25*GAM*T3(E)/DT+.5*G3(E)*YD/DX)/( .25*T3(E)/DT+.5*G3(
*E)/DX+ALFA)
F4(E)=F=(HET=.25*GAM*T4(E)/DT+.5*G4(E)*YD/DX)/( .25*T4(E)/DT+.5*G4(
*E)/DX+ALFA)
PAY1(E)=.25*GAM*T1P(E)/DT+.50*G1P(E)*YD/DX
PAY2(E)=.25*GAM*T2P(E)/DT+.50*G2P(E)*YU/DX
PAY3(E)=.25*GAM*T3P(E)/DT+.50*G3P(E)*YD/DX
PAY4(E)=.25*GAM*T4P(E)/DT+.50*G4P(E)*YD/DX
PA1(E)=(.25*T1P(E)/DT+.5*G1P(E)/DX)*(HET=.25*GAM*T1(E)/DT+.5*YD*G1

```


CCCCCCCCCCCCCCCC

CCCCCCCC

C

С С С С С С С С С С

CCCCC

C

2

5

2

5

2

1

1

1

1

1

```

C      READ(5,402)FINS6(I),FFING(I),CINFG(I),FINSPI(I),FFINP(I),CINFP(I),
      *CGTR1(I),CGTR2(I)
402  FORMAT(8F5,0)
      IF(UNIT.EQ.1)GO TO 8003
      FINS6(I)=FINS6(I)*0.03937
      FFING(I)=FFING(I)*0.03937
      FINSPI(I)=FINSPI(I)*0.03937
      FFINP(I)=FFINP(I)*0.03937
8003  CONTINUE
C
C      K1,K2,K3,K4,K5,K6 ARE IDENTIFICATION NUMBERS OF SIX IMMEDIATELY
C      UPSTREAM INLETS
C      OR1,OR2,OR3,OR4,OR5,OR6 ARE CARRY-OVER DISTRIBUTION FACTORS
C
      READ(5,502)K1(I),K2(I),K3(I),K4(I),K5(I),K6(I),      OR1(I),OR2(I)
      *OR3(I),OR4(I),OR5(I),OR6(I)
502  FORMAT(6I5,6F5,0)
      NNN=N(I)
      DO 1 J=2,NNN
C
C      NL,OS,OK RESPECTIVELY ARE THE LENGTH SLOPE AND SURFACE ROUGHNESS
C      OF SUBCATCHMENT STRIP
C      FINSHL,FFINAL,CINFLT ARE THE INFILTRATION PARAMETERS FOR SUBCATCHMENT
C      YIND IS INITIAL DEPTH OF WATER IN SUBCATCHMENT
C      C01 AND C02 ARE INITIAL CONCENTRATIONS OF THE 1ST AND 2ND
C      POLLUTANTS IN SUBCATCHMENT
C      ND IS NUMBER OF COMPUTATIONAL GRID POINTS ALONG SUBCATCHMENT STRIP
C
      READ(5,103)NL(I,J),OS(I,J),OK(I,J),FINSHL(I,J),FFINAL(I,J),CINFLT(
      *I,J),YIND(I,J),C01(I,J),C02(I,J),ND(I,J)
103  FORMAT(9F5,0,15)
      IF(UNIT.EQ.1)GO TO 8004
      NL(I,J)=NL(I,J)*3.28
      OK(I,J)=OK(I,J)*3.28
      FINSHL(I,J)=FINSHL(I,J)*0.03937
      FFINAL(I,J)=FFINAL(I,J)*0.03937
      YIND(I,J)=YIND(I,J)*3.28
8004  CONTINUE
      1 CONTINUE
      IF(I.EQ.ITOTAL)GO TO 712
      I=I+1
      GO TO 36
C
C      FOLLOWING IS INPUT DATA FOR SEWER NODES
C
712  DO 702 JUNC=1,JTOTAL
C
C      NSW AND SWRTYP ARE IDENTIFICATION NUMBER AND TYPE OF SEWER NODE
C      BYSFLO IS RATE OF BASE FLOW INTO SEWER NODE
C      CJ1 AND CJ2 ARE CONCENTRATIONS OF TWO POLLUTANTS IN BASE FLOW
C      INLET'S ARE IDENTIFICATION NUMBERS OF GUTTER INLETS DISCHARGING
C      INTO SEWER NODE
C
      READ(5,703)NSW(JUNC),SWRTYP(JUNC),BYSFLO(JUNC),CJ1(JUNC),CJ2(JUNC)
      *INLET(JUNC,INL),INL=1,10)
      IF(UNIT.EQ.1)GO TO 8005
      BYSFLO(JUNC)=BYSFLO(JUNC)*3.28*3.28*3.28
8005  CONTINUE
      202 FORMAT(3F10,0,15)
      703 FORMAT(15,4F5,0,1015)
      702 CONTINUE

```

```

      DO 900 JK=1,7
      DO 113 I=1,ITOTAL
      GO TO(901,902,903,904,905,906,907),JK
901  IF(K1(I).EQ.0)GO TO 908
      GO TO 113
902  IF(K2(I).EQ.0.AND.K1(I).NE.0)GO TO 908
      GO TO 113
903  IF(K3(I).EQ.0.AND.K2(I).NE.0)GO TO 908
      GO TO 113
904  IF(K4(I).EQ.0.AND.K3(I).NE.0)GO TO 908
      GO TO 113
905  IF(K5(I).EQ.0.AND.K4(I).NE.0)GO TO 908
      GO TO 113
906  IF(K6(I).EQ.0.AND.K5(I).NE.0)GO TO 908
      GO TO 113
907  IF(K6(I).NE.0)GO TO 908
      GO TO 113
908  CONTINUE
      2 TYM=0.0
      L1 = 0
      L2=NDTM
      L3=1
      GTR=GTRTYP(I)
      IF(GTR.EQ.0.)GO TO 500
      IF(GTR.EQ.2.0)GO TO 888
      SINTH=SIN(T8A(I))
      COSTH=COS(T8A(I))
      COTTH=COSTH/SINTH
      TANTH=TAN(T8A(I))
888  DX=GL(I)/(N(I)-1)
      XN=RAFCOF(I)
      HHHH=H(I)
      NNN=N(I)
      SS=SG(I)
      U=B(I)
      GD=U*COTTH
      DO 3 I1=1,NNN
      OD(I1)=0.0
      3 YD(I1)=YING(I)
      COF=1.49*SQRT(SS)/XN
500  DT=DTM
      TYM=TYM+DT
      L1=L1+1
      L2=L2-1
      IF(GTR.NE.0.)GO TO 2666
      CALL UPS80
      QP=QUPST
      CALL STROUT
      GO TO 2678
2666 AYSE=0.
      FATMA=0.
      GUL=0.
      ROK1=0.
      BOK2=0.
2875 DO 5 J1=1,NNN
      IF(J1.NE.1)GO TO 77
1000 CALL UPS80
      QN(J1)=QUPST
      YN(J1)=YUPST
      IF(GTR.EQ.1.0)AUPST=0.5*COTTH*YUPST*YUPST
      IF(GTR.EQ.2.0)AUPST=YUPST*B(I)
      GO TO 5

```

```

77 IF(J1.NE.2)GO TO 7
   FFIN=FFINP(I)
   FINS=FINSF(I)
   CINF=CINFP(I)
   IRYN=IRNZON(I)
   CALL RASNIN
   QL1=QRSI*PL(I)
   FFIN=FFING(I)
   FINS=FINSF(I)
   CINF=CINFG(I)
   IRYN=IRNZON(I)
   CALL RASNIN
   QL3=QRSI
   GO TO 8
7 IF(OK(I,J1).EQ.OK(I,J1-1).AND.OS(I,J1).EQ.OS(I,J1-1).AND.NL(I,J1).
  *EQ.OL(I,J1-1))GO TO 9
   GO TO 8
9 IF(FFINAL(I,J1).EQ.FFINAL(I,J1-1).AND.FINSHL(I,J1).EQ.FINSHL(I,J1
  *-1).AND.CINFLT(I,J1).EQ.CINFLT(I,J1-1))GO TO 1200
   GO TO 8
1200 IF(CO1(I,J1).EQ.CO1(I,J1-1).AND.CO2(I,J1).EQ.CO2(I,J1-1))GO TO 12
8 FK=OK(I,J1)
   SD=OS(I,J1)
   NLL=OL(I,J1)
   FFIN=FFINAL(I,J1)
   FINS=FINSHL(I,J1)
   CINF=CINFLT(I,J1)
   NDV=ND(I,J1)
   YDIN=YIND(I,J1)
   IRYN=IRNZON(I)
   CONIN1=CO1(I,J1)
   CONIN2=CO2(I,J1)
   CALL OVLFLD
   QL2=OVERL
12 QLAT=QL1+QL2+QL3*U
   FLOLAT=QLAT
   YA=YD(J1-1)
   YB=YD(J1)
   YD=YN(J1-1)
   GAM=YD-YA-YB
   MT=0
   MTT=0
   YD=YD
   IF(J1.EQ.2)YD=YD+(DX*XN*QLAT/SQRT(100.*SS))*(.3/.8)
   DELYD=YD-YING(I)
   IF(GTR.EQ.1.0)GO TO 19
   IF(YD.GT.HHHH)GO TO 70
   RET=QLAT-.25*T3(YD)*GAM/UT+.5*YD*G3(YD)/DX
   ALFA=.25*T3(YD)/DT+.5*G3(YD)/DX
   GO TO 23
70 ALFA=.25*T4(YD)/DT+.5*G4(YD)/DX
   BET=QLAT-.25*T4(YD)*GAM/DT+.5*YD*G4(YD)/DX
23 IF(GTR.EQ.1.0)GO TO 19
   IF(YD.GT.HHHH)GO TO 21
   CONVER=F3(YD)*F3PP(YD)/(F3P(YD)**2)
   CONVER=ABS(CONVER)
   GO TO 20
21 CONVER=F4(YD)*F4PP(YD)/(F4P(YD)**2)
   CONVER=ABS(CONVER)
20 IF(CONVER.LT.1.0)GO TO 22
   GO TO 32
19 IF(YD.GT.GD)GO TO 80

```



```

ALFA=.25*T1(YD)/DT+.5*G1(YD)/DX
RET=QLAT-.25*T1(YD)*GAH/DT+.5*YD*G1(YD)/DX
GO TO 89
80 ALFA=.25*T2(YD)/DT+.5*G2(YD)/DX
RET=QLAT-.25*T2(YD)*GAH/DT+.5*YD*G2(YD)/DX
89 IF(YD.GT.GD)GO TO 31
CONVER=F1(YD)*F1PP(YD)/(F1P(YD)**2)
CONVER=ABS(CONVER)
GO TO 30
31 CONVER=F2(YD)*F2PP(YD)/(F2P(YD)**2)
CONVER=ABS(CONVER)
30 IF(CONVER.LT.1.0)GO TO 22
32 IF(MTT.NE.0)GO TO 35
YD=YD-DELYD/20.
IF(YD.LT.YING(I))YD=YING(I)
MT=MT+1
IF(MT.LT.20)GO TO 23
35 IF(MTT.EQ.0)YD=YD
IF(MTT.EQ.0.AND.J1.EQ.2)YD=YD+(DX*XM*QLAT/SQRT(100.**55))**(.3/.8.)
YD=YD+YD/5.0
MTT=MTT+1
IF(MTT.LE.99)GO TO 23
22 DO 37 M1=1.40
YD=ABS(YD)
IF(GTR.EQ.1.0)GO TO 38
IF(YD.GT.HHHH)GO TO 39
FUNC=F3(YD)
FUNC=F3P(YD)
GO TO 40
38 IF(YD.GT.GD)GO TO 41
FUNC=F1(YD)
FUNC=F1P(YD)
GO TO 40
41 FUNC=F2(YD)
FUNC=F2P(YD)
GO TO 40
39 FUNC=F4(YD)
FUNC=F4P(YD)
40 Y1=YD-FUNC/FUNCP
IF(ABS(Y1-YD).LE.0.00001)GO TO 42
37 YD=Y1
42 YC=Y1
45 YN(J1)=YC
AYSE=AYSE+QL1
FATMA=FATMA+QL2
GUL=GUL+QL3
BOK1=BOK1+COVER1+QL2
BOK2=BOK2+COVER2+QL2
IF(J1.NE.NNN)GO TO 5
YP=YC
IF(GTR.EQ.2.)GO TO 301
IF(YP.GT.GD)GO TO 302
QP=Q1(YC)
A=A1(YC)
VP=QP/A
T=T1(YC)
GO TO 303
302 QP=Q2(YC)
A=A2(YC)
VP=QP/A
T=T2(YC)
GO TO 303

```

```

301 IF(YP.GT.HHHH)GO TO 304
   OP=Q3(YC)
   A=A3(YC)
   VP=OP/A
   T=T3(YC)
   GO TO 303
304 OP=Q4(YC)
   A=A4(YC)
   VP=OP/A
   T=T4(YC)
303 TOTAL=AYSE+FATMA+GUL
   C2LAT=RDK2/TOTAL
   C1LAT=RDK1/TOTAL
   ZIM=TOTAL/(NNN-1)
   IF(L1.EQ.1)CIB1=CGTR1(I)
   IF(L1.EQ.1)CIB2=CGTR2(I)
   CIC1=(C1LAT+CIB1*A/DT+CUPST1*OP/GL(I))/(A/DT+OP/GL(I)+ZIM)
   CIC2=(C2LAT+CIB2*A/DT+CUPST2*OP/GL(I))/(A/DT+OP/GL(I)+ZIM)
   CIB1=CIC1
   CIB2=CIC2
3303 CALL DOWNBOU
   5 CONTINUE
   DO 55 J2=1,NNN
   55 YD(J2)=YN(J2)
2678 IF(TYM.LT.TYMEND)GO TO 500
   LIMIT=L3
   113 CONTINUE
   900 CONTINUE
   CALL SWRJNT
   STOP
   END
   SUBROUTINE UPSBO
C
C   THIS SUBROUTINE COMPUTES UPSTREAM BOUNDARY CONDITION AT EACH
C   TIME LEVEL
C
COMMON/Z5/SG,RAFCOF,YING,COTTH,SINTH,COSTH,TANTH,YUPST,QUPST
COMMON/Z6/K1,K2,K3,K4,K5,OR1,OR2,OR3,OR4,OR5,OR6,K5
COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
* CIC1,CIC2
COMMON/Z8/G,L1,TYM
DIMENSION SG(100),RAFCOF(100),YING(100)
DIMENSION K1(100),K2(100),K3(100),K4(100),K5(100),K6(100)
DIMENSION DC1(6),DC2(6)
DIMENSION OR1(100),OR2(100),OR3(100),OR4(100),OR5(100),OR6(100)
DIMENSION UPINF(6),CGTR1(100),CGTR2(100)
DIMENSION CCC1(100,100),CCC2(100,100)
DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100),NPR(100),NGTR(
* J00)
DO 1 MM=1,6
  IF(MM.EQ.1)MIN=K1(I)
  IF(MM.EQ.2)MIN=K2(I)
  IF(MM.EQ.3)MIN=K3(I)
  IF(MM.EQ.4)MIN=K4(I)
  IF(MM.EQ.5)MIN=K5(I)
  IF(MM.EQ.6)MIN=K6(I)
  IF(MIN.EQ.0)UPINF(MM)=0.0
  IF(MIN.EQ.0)DC1(MM)=CGTR1(I)
  IF(MIN.EQ.0)DC2(MM)=CGTR2(I)
  IF(MIN.EQ.0)GO TO 1
  HAL=2
69 IF(TYM-TIME(HAL))112,111,110

```

```

110 MAL=MAL+1
    GO TO 69
111 UPINF(MH)=QUP(MIN,MAL)
    DC1(MH)=CCC1(MIN,MAL)
    DC2(MH)=CCC2(MIN,MAL)
    GO TO 1
112 UPINF(MH)=QUP(MIN,MAL-1)+(QUP(MIN,MAL)-QUP(MIN,MAL-1))/(TIME(MAL)-
    *TIME(MAL-1))*(TYM-TIME(MAL-1))
    RATIO=(TYM-TIME(MAL-1))/(TIME(MAL)-TIME(MAL-1))
    DC1(MH)=CCC1(MIN,MAL-1)+(CCC1(MIN,MAL)-CCC1(MIN,MAL-1))*RATIO
    DC2(MH)=CCC2(MIN,MAL-1)+(CCC2(MIN,MAL)-CCC2(MIN,MAL-1))*RATIO
1 CONTINUE
    DISCH=(1.0)*(OR1(1)*UPINF(1)+OR2(1)*UPINF(2)+OR3(1)*UPINF(3)+OR4(1)
    *)*UPINF(4)+OR5(1)*UPINF(5)+OR6(1)*UPINF(6))
    IF(DISCH.EQ.0.)CUPST1=CGTR1(1)
    IF(DISCH.EQ.0.)CUPST2=CGTR2(1)
    IF(DISCH.EQ.0.0)GO TO 200
    DIS1=OR1(1)*UPINF(1)+DC1(1)+OR2(1)*UPINF(2)+DC1(2)+OR3(1)*UPINF(3)
    **DC1(3)+OR4(1)*UPINF(4)+DC1(4)+OR5(1)*UPINF(5)+DC1(5)+OR6(1)*UPINF
    *(6)+DC1(6)
    DIS2=OR1(1)*UPINF(1)+DC2(1)+OR2(1)*UPINF(2)+DC2(2)+OR3(1)*UPINF(3)
    **DC2(3)+OR4(1)*UPINF(4)+DC2(4)+OR5(1)*UPINF(5)+DC2(5)+OR6(1)*UPINF
    *(6)+DC2(6)
    CUPST1=DIS1/DISCH
    CUPST2=DIS2/DISCH
200 IF(GTRTYP(1).EQ.0.0)GO TO 4
    IF(GTRTYP(1).EQ.2.0)GO TO 2
    YUPST=(DISCH*RAFCUF(1)*CUTTH/(.745*SQRT(SG(I))))**(.375)*((1.+COST
    1H)/(0.5+SINTH))**(.25)
    GO TO 3
2 YUPST=(DISCH*RAFCUF(1)/(1.49*8(1)*SQRT(SG(I))))**(.375)
3 IF(YUPST.LT.YING(1))YUPST=YING(1)
4 OUPST=DISCH
    RETURN
    END
    SUBROUTINE DOWBOU

```

C
C
C

THIS SUBROUTINE COMPUTES FLOW INTO GUTTER INLET AND CARRY-OVER
AT EACH TIME LEVEL

```

COMMON/Z4/QCACH,TBA,GRBTYP,W,GRBL,GD,T,VP,YP,OP,FLOLAT,OPR
COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
*CCC1,CCC2
COMMON/Z8/G,L1,TYM
COMMON/Z10/L2,L3,NGTH,NGTR
DIMENSION CGTR1(100),CGTR2(100)
DIMENSION CCC1(100,100),CCC2(100,100)
DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100),OPR(100),NGTR(
*100)
DIMENSION QCACH(100,100),TBA(100),GRBTYP(100),W(100),GRBL(100)
TIME(1)=0.
QUP(1,1)=0.0
IF(GRBTYP(1).EQ.0.0)GO TO 20
IF(GRBTYP(1).EQ.1.0.OR.GRBTYP(1).EQ.2.0)COF=1.0
IF(GRBTYP(1).EQ.3.0)COF=0.4
IF(GRBTYP(1).EQ.4.0)COF=0.8
IF(GRBTYP(1).EQ.5.0)COF=0.9
IF(GTRTYP(1).EQ.2.0)GO TO 14
DEP=W(1)/TAN(TBA(1))
IF(YP.GT.DEP)GO TO 13
WL=T*OPR(1)/SIN(TBA(1))
YM=YP/2.

```

```

      GO TO 12
13  WL=W(I)*DPR(I)/SIN(TBA(I))
    YH=YP-DEP/2.
    GO TO 12
14  WL=W(I)*DPR(I)
    YH=YP
12  WFL=3.*CDF*WL*(YH+VP*VP/(2.*G))*1.5
    IF(WFL.LE.QP)GO TO 11
    WFL=0.6*WL*GR6L(I)*SQRT(YH+VP*VP/(2.*G))
    IF(WFL.GT.QP)WFL=QP
11  FLOWIN=WFL*GR6L(I)*FLOLAT
    CARYOV=QP-WFL
    GO TO 21
20  FLOWIN=0.
    CARYOV=QP
21  IF(L2.NE.0)GO TO 31
    L3=L3+1
    L2=NDTH
    QUP(NGTR(I),L3)=CARYOV
    QCACH(NGTR(I),L3)=FLOWIN
    CCC1(NGTR(I),L3)=CIC1
    CCC2(NGTR(I),L3)=CIC2
    TIME(L3)=TYH
10  CONTINUE
31  RETURN
    FND
    SUBROUTINE RASNIN
C
C   THIS SUBROUTINE COMPUTES DIRECT LATERAL INFLOW FROM RAIN,SNOW,
C   AND INFILTRATION
C
    COMMON/Z1/TEMP,IRYN,RYNST,RYNEND,DAYRYN,TH,RAIN
    COMMON/Z2/FFIN,FINS,CINF,QRAIN,QRSI
    COMMON/ZB/G,L1,TYM
    DIMENSION RYNST(5),RYNEND(5),TH(5,100),RAIN(5,100),DAYRYN(5)
    IF(TYM.LT.RYNST(IRYN),OR.TYM.GT.RYNEND(IRYN))GO TO 1
    J2=1
69  IF(TYM-TH(IRYN,J2))112,111,110
110 J2=J2+1
    GO TO 69
111 QRAIN=RAIN(IRYN,J2)/(12.0*3600.)
    GO TO 2
112 QRAIN=RAIN(IRYN,J2-1)+(TYM-TH(IRYN,J2-1))*(RAIN(IRYN,J2)-RAIN(IRYN
    ,J2-1))/(TH(IRYN,J2)-TH(IRYN,J2-1))
    QRAIN=QRAIN/(3600.*12.0)
    GO TO 2
1  QRAIN=0.0
    IF(TYM.LT.RYNST(IRYN))QINFLT=0.0
    IF(TYM.LT.RYNST(IRYN))GO TO 5
2  FAFA=FINS/(12.0*3600.0)
    IF(QRAIN.LE.FAFA)GO TO 4
    ARG=-CINF*(TYM-RYNST(IRYN))/(3600.)
    QINFLT=FFIN+(FINS-FFIN)*EXP(ARG)
    QINFLT=QINFLT/(3600.*12.0)
    GO TO 5
4  QINFLT=QRAIN
5  QSNOW=0.007*DAYRYN(IRYN)*(TEMP-32.)/(24.*3600.*12.0)
    IF(TEMP.LT.32.0)QSNOW=0.
    QRSI=QRAIN-QINFLT+QSNOW
    RETURN
    FND
    SUBROUTINE OVLFLD

```

C
C
C
C

THIS SUBROUTINE COMPUTES THE QUANTITY AND QUALITY OF FLOW IN
SUBCATCHMENT STRIPS

```

COMMON/Z1/TEMP,IRYN,RYNST,RYNEND,DAYRYN,TH,RAIN
COMMON/Z2/FFIN,FINS,CINF,QRAIN,QRSI
COMMON/Z20/CUNIN1,CUNIN2,COVER1,COVER2
COMMON/Z3/XNU,C1,FK,SO,OLL,NOV,YOIN,DT,QOVERL
COMMON/Z8/G,LI,LYH
DIMENSION YOLD(11),YNEW(11),QOLD(11),QNEW(11)
DIMENSION RYNST(5),RYNEND(5),TH(5,100),RAIN(5,100),DAYRYN(5)
GL(E)=3.*ALF*E**2
GLP(E)=6.*ALF*E
GLPP(E)=6.*ALF
FL(E)=E-(BET+RAT*YD+GL(E))/(GAM+RAT*GL(E))
FUNC1(E)=YD+GLP(E)/(GL(E)+GAM/RAT)*(-1.)
FUNCPL(E)=(YD+GLPP(E)+(GL(E)+GAM/RAT)-YD+GLP(E)**2)/(GL(E)+GAM/RAT
1)**2*(-1.)
PAY(E)=RAT*GLP(E)*(BET+YD+RAT*GL(E))
PAYP(E)=RAT*(BET+RAT*YD+GL(E))*GLPP(E)+YD*(RAT+GLP(E))**2
PAYD(E)=(GAM+RAT*GL(E))*2
PAYDP(E)=2.*RAT*GLP(E)*(GAM+RAT*GL(E))
FLP(E)=1.0+FUNC1(E)+PAY(E)/PAYD(E)
FLPP(E)=FUNCPL(E)+(PAYP(E)+PAYD(E)-PAYDP(E)+PAY(E))/(PAYD(E))**2
QL(E)=ALF*E**3
GT(E)=SQRT(8.*G*SO*E)*(3.*ALOG10(2.*E/FK)+3.47)
GTP(E)=SQRT(8.*G*SO/E)*(1.5*ALOG10(2.*E/FK)+3.04)
GTPP(E)=SQRT(8.*G*SO/E**3)*(0.75*ALOG10(2.*E/FK)-0.85)
FT(E)=E-(BET+RAT*YD+GT(E))/(GAM+RAT*GT(E))
FUNCIT(E)=YD+GTP(E)/(GT(E)+GAM/RAT)*(-1.)
FUNCPT(E)=(YD+GTPP(E)+(GT(E)+GAM/RAT)-YD+GTP(E)**2)/(GT(E)+GAM/RAT
1)**2*(-1.)
PAT(E)=RAT+GTP(E)*(BET+YD+RAT+GT(E))
PATP(E)=RAT*(BET+RAT*YD+GT(E))*GTPP(E)+YD*(RAT+GTP(E))**2
PATD(E)=(GAM+RAT*GT(E))*2
PATDP(E)=2.*RAT+GTP(E)*(GAM+RAT+GT(E))
FTP(E)=1.0+FUNCIT(E)+PAT(E)/PATD(E)
FTPP(E)=FUNCPT(E)+(PATP(E)+PATD(E)-PATDP(E)+PAT(E))/(PATD(E))**2
QT(E)=SQRT(8.*G*SU*E**3)*(2.*ALOG10(2.*E/FK)+1.74)
REYCR(E)=C1*(2.*ALOG10(2.*E/FK)+1.74)**2
IF(OLL.EQ.0.0)QOVERL=0.0
IF(OLL.EQ.0.0)GO TO 38
CALL RASIN
QOL=QRSI
DOX=OLL/(NOV-1)
ALF=8.*G*SO/(C1*XNU)
RAT=DT/DOX
IF(LI.NE.1)GO TO 2
FLOW=1.
DO 1 INV=1,NOV
1 YOLD(INV)=YOIN
2 DO 3 KDV=1,NOV
  IF(KDV.EQ.1)GO TO 4
  YOA=YOLD(KDV-1)
  YOD=YNEW(KDV-1)
  YD=YOD
  YOB=YOLD(KDV)
  IF(FLOW.EQ.2.0.AND.YOB.GT.FK.AND.YOD.GT.FK)GO TO 31
  BET=2.*QOL*DT-(YOD-YOA-YOB)*RAT+YOD*GL(YOD)
  GAM=1.*RAT*GL(YOD)
  MOT=0
  MOTT=0

```



```

Y0=((ONEW(KOV-1)+QOL*DOX)/ALF)**0.33
D1Y0=(Y0-YOIN)/20.
D2Y0=Y0/10.
YY0=Y0
7 CONVER=FL(Y0)*FLPP(Y0)/(FLP(Y0)**2)
CONVER=ARS(CONVER)
IF(CONVER.LT.1.0)GO TO 5
IF(MOT.NE.0)GO TO 6
Y0=Y0-D1Y0
MOTT=MOTT+1
IF(MOTT.LE.20)GO TO 7
6 IF(MOT.EQ.0)Y0=YY0
Y0=Y0+D2Y0
MOT=MOT+1
IF(MOT.LE.75)GO TO 7
GO TO 31
5 DO 9 M1=1.50
Y0=ARS(Y0)
Y1=Y0-FL(Y0)/FLP(Y0)
IF(ABS(Y0-Y1).LE.0.00001)GO TO 10
9 Y0=Y1
10 YOC=Y1
IF(YOC.LT.YOIN)YOC=YOIN
YNEW(KOV)=YOC
QNEW(KOV)=OL(YOC)
REYN0=QNEW(KOV)/XNU
IF(REYN0.LT.REYCR(YOC).OR.YOD.LT.FK)GO TO 3
IF(REYN0.LT.REYCR(YOC).OR.YOB.LT.FK)GO TO 3
FLN0=2.
31 BET=2.0+QOL*DT-(YOD-YOA-YOB)*RAT*YOD*GT(YOD)
GAM=1.+RAT*GT(YOD)
MAT=0
MATT=0
Y0=YOB+QOL*DT
D1Y0=(Y0-YOIN)/20.
D2Y0=Y0/10.
YY0=Y0
37 CONVER=FT(Y0)*FTPP(Y0)/(FTP(Y0)**2)
CONVER=ARS(CONVER)
IF(CONVER.LT.1.)GO TO 35
IF(MAT.NE.0)GO TO 36
Y0=Y0-D1Y0
IF(Y0.LT.FK)Y0=FK
MATT=MATT+1
IF(MATT.LE.20)GO TO 37
36 IF(MAT.EQ.0)Y0=YY0
Y0=Y0+D2Y0
MAT=MAT+1
IF(MAT.LE.75)GO TO 37
WRITE(6,8)
8 FORMAT(2X,'OVERLAND ITERATION FAILS')
STOP
35 DO 39 M2=1.50
Y0=ARS(Y0)
Y1=Y0-FT(Y0)/FTP(Y0)
IF(ABS(Y0-Y1).LE.0.00001)GO TO 40
39 Y0=Y1
40 YOC=Y1
IF(YOC.LT.YOIN)YOC=YOIN
YNEW(KOV)=YOC
QNEW(KOV)=QT(YOC)
GO TO 3

```

```

4 YNEW(1)=YDIN
  QNEW(1)=0.0
3 CONTINUE
  IF(L1.EQ.1)CONB1=CONIN1
  IF(L1.EQ.1)CONB2=CONIN2
  QOVERL=QNEW(NOV)
  COVER1=(CONB1*YNEW(NOV)/DT+CONIN1*QOVERL/OLL)/(YNEW(NOV)/DT+QOVERL
  * /OLL+QOL)
  COVER2=(CONB2*YNEW(NOV)/DT+CONIN2*QOVERL/OLL)/(YNEW(NOV)/DT+QOVERL
  * /OLL+QOL)
  DO 47 JOV=1,NOV
    CONB1=COVER1
    CONB2=COVER2
47 YOLD(JOV)=YNEW(JOV)
38 RETURN
  END
  SUBROUTINE STRUT
C
C THIS SUBROUTINE COMPUTES FLOW INTO INLETS WHERE THERE IS NO
C DOWNSTREAM GUTTERS
C
COMMON/Z4/QCACH,TBA,GRBTYP,W,GRBL,GD,T,VP,Y,P,OP,FLOLAT,OPR
COMMON/Z3/XNU,C1,FK,SD,OLL,NOV,YDIN,DT,QOVERL
COMMON/Z8/G,L1,TYM
COMMON/Z10/L2,L3,NDTM,NGTR
COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
  * C1C1,C1C2
COMMON/Z11/GL
DIMENSION QCACH(100,100),GRBTYP(100),TBA(100)
DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100)
DIMENSION CGTR1(100),CGTR2(100)
DIMENSION CCC1(100,100),CCC2(100,100)
DIMENSION GL(100),W(100),NGTR(100),GRBL(100),OPR(100)
IF(L1.NE.1)GO TO 1
AREA=W(1)*GRBL(1)*UPR(1)
XK=-(0.80*AREA*SQRT(2.*G))/2.
XC1=GL(1)/DT
XI1=0.
XH1=0.
XQ1=0.
1 X12=QP
XC2=.5*(XI1+XI2+XQ1)+XC1*XH1
DISC=XK**2+4.0*XC1*XC2
XH0=(+XK+SQRT(DISC))/(2.*XC1)
IF(XH0.LT.0.0)XH0=0.
XH2=XH0**2
XQ2=-2.*XK*XH0
IF(L2.NE.0)GO TO 2
L2=NDTM
L3=L3+1
QCACH(NGTR(1),L3)=XQ2
IF(XQ2.EQ.0.)CCC1(NGTR(1),L3)=0.
IF(XQ2.EQ.0.)CCC2(NGTR(1),L3)=0.
IF(XQ2.EQ.0.)GO TO 2
CCC1(NGTR(1),L3)=CUPST1*X12/XQ2
CCC2(NGTR(1),L3)=CUPST2*X12/XQ2
2 XQ1=XQ2
XH1=XH2
XI1=XI2
RETURN
  END
  SUBROUTINE SWRJNT

```

```

C
C      THIS SUBROUTINE COMPUTES THE QUANTITY AND QUALITY OF DIRECT INFLOW
C      TO SEWER NODES
C
COMMON/Z4/QCACH,TBA,GR8TYP,W,GR8L,GD,T,VP,YP,QP,FLOLAT,OPR
COMMON/Z7/I,GTRTYP,B,TIME,QUP,CGTR1,CGTR2,CUPST1,CUPST2,CCC1,CCC2,
* CIC1,CIC2
COMMON/Z8/G,L1,TYM
COMMON/Z9/JTOTAL,INLET,LIMIT,TYHEND,DTM,NSW,SWRTYP,BYSFLO
COMMON/Z12/CJ1,CJ2
DIMENSION CJ1(100),CJ2(100)
DIMENSION ITYM(100),QDIS(100)
DIMENSION INLET(100,10),NSW(100),SWRTYP(100),BYSFLO(100)
DIMENSION QCACH(100,100),TBA(100),GR8TYP(100),W(100),GR8L(100)
DIMENSION COB1(100),COB2(100),CGTR1(100),CGTR2(100)
DIMENSION CCC1(100,100),CCC2(100,100)
DIMENSION GTRTYP(100),B(100),TIME(100),QUP(100,100),OPR(100),NGTR(
* 100)
IHD=TYHEND
IDTM=DTM
ISTP=IHD+IDTM
DO 1 JJ=1,JTOTAL
WRITE(6,200)
200 FORMAT(2X,'*****')
WRITE(6,80)NSW(JJ)
80 FORMAT(1X,'FLOW INTO SEWER JUNCTION=',I5,'*****')
WRITE(6,200)
WRITE(6,81)
81 FORMAT(///2X,'TIME',3X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POLL
* UTANT')
WRITE(6,86)
86 FORMAT(1X,'(SEC)',6X,'(CFS)',6X,'CONCENTRATION',3X,'CONCENTRATION'
*)
TIME(1)=0.
ITYM(1)=0
QDIS(1)=BYSFLO(JJ)
COB1(1)=CJ1(JJ)
COB2(1)=CJ2(JJ)
DO 2 NN=2,LIMIT
ITYM(NN)=TIME(NN)
6 DIS=0.
CIS1=0.
CIS2=0.
DO 3 MM=1,10
K=INLET(JJ,MM)
IF(K.EQ.0)GO TO 15
CIS1=CIS1+QCACH(K,NN)+CCC1(K,NN)
CIS2=CIS2+QCACH(K,NN)+CCC2(K,NN)
3 DIS=DIS+QCACH(K,NN)
15 QDIS(NN)=DIS+BYSFLO(JJ)
COB1(NN)=(CIS1+BYSFLO(JJ)+CJ1(JJ))/QDIS(NN)
COB2(NN)=(CIS2+BYSFLO(JJ)+CJ2(JJ))/QDIS(NN)
2 CONTINUE
14 NL=LIMIT+1
ITYM(NL)=ISTP
QDIS(NL)=BYSFLO(JJ)
IF(SWRTYP(JJ).EQ.2.0)GO TO 20
WRITE(7,21)NSW(JJ),IHD,BYSFLO(JJ)
21 FORMAT('JFBD',1X,I5,1X,'0',110,1X,F10.2)
GO TO 22
20 WRITE(7,23)NSW(JJ),IHD,BYSFLO(JJ)
23 FORMAT('FRD',1X,I5,1X,'0',110,1X,F10.2)

```

```

22 WRITE(7,25)(ITYH(MH),QDIS(MH),MH=1,NL)
25 FORMAT(15,1X,F5.2,1X,15,1X,F5.2,1X,15,1X,F5.2,1X,15,1X,F5.2,1X,15,
  *1X,F5.2,1X,15,1X,F5.2)
  IF(SWRTYP(JJ).EQ.2.0)GO TO 30
  WRITE(7,31)
31 FORMAT('JEND')
  GO TO 100
30 WRITE(7,32)
32 FORMAT('FEND')
100 DO 82 MAMA=1,LIMIT
  WRITE(6,83)TIME(MAMA),QDIS(MAMA),COB1(MAMA),COB2(MAMA)
83 FORMAT(F7.2,2X,F8.2,9X,F6.2,9X,F6.2)
82 CONTINUE
  1 CONTINUE
  RETURN
  END

```

APPENDIX C

DATA PREPARATION AND LISTING OF COMPUTER PROGRAM FOR ILLINOIS SEWER SYSTEM WATER QUALITY MODEL

The Illinois Sewer System Water Quality Model is programmed in Fortran IV language. The input to the computer program includes the depth and discharge hydrographs at the entrance and exit of each sewer and the volume of water at each storage junction at given times as provided by the output of the ISS model. In addition, the direct inflow hydrographs and pollutographs for up to two pollutants to the sewer junctions as obtained from surface runoff computations and the sewer system layout are also input to the program for runoff quality routing in the sewer system. The output from the computer program are the pollutographs at the sewer system outlet, at the storage junctions and at the entrance and exit of all sewers. The storage requirement for the computer program in its present form is 300k bytes. A description of the computer program has been presented in Sec. VIII-3 and the flow chart in Fig. 29.

(a) Data Preparation

The input data to the computer program can be in either English or SI units. The input concentration of the pollutants can be in either ppm or mg/l and the output will be in the corresponding unit. The input data deck consists of the following three sets in the order of presentation. The first samples shown in Figs. C-1 to C-3 correspond to the values given as respective examples in the Entry in the description.

Set 1. General description of sewer system and input data — This set consists of one card as follows:

Card	Column	Format	Entry
1-1	1-5	I5	Total number of sewers in the system; e.g., 18
1-1	6-10	I5	Total number of reservoir-type junctions in the system; e.g., 5
1-1	11-15	I5	Number of points used to describe each of the discharge and stage hydrographs at the entrance and exit of each sewer*, maximum 100; e.g., 12
1-1	16-20	I5	Number of points used to describe direct inflow hydrographs at each of the sewer nodes*; e.g., 10

*Actually most likely the input hydrographs will have different time length and some may be described by fewer points than the others. However, a common number of points are required for all the hydrographs for the purpose of input data preparation. Therefore, for short duration hydrographs the user may include a number of points belonging to the tail part of the hydrographs.

Example	Entry														
	Col.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	18	5	12	10	8	120.									
2	25	8	10	8	19	60.									
3	6	3	15	15	12	30.									

Fig. C-1. Three examples of data Set 1 for sewer water quality model

Example	Entry														
	Col.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	10	15	400.	1.75	.001	.05	.002	.06							
	0.	1.0	0.4	1.	.88	210.	4.5	.86	2.8	.97	357.	7.0	1.17	5.4	1.23
	740.	4.7	1.0	4.8	1.61	4700.	1.0	.40	1.0	.98					
2	28	100	800.	3.5	.01	.005	.01	.005							
	0.	1.4	.28	1.4	.89	396.	7.80	1.34	2.1	1.11	990.	2.7	2.01	3.1	2.1
	1369.	1.4	1.74	4.2	1.77										

Fig. C-2. Two examples of data Set 2 for sewer water quality model

Example	Entry														
	Col.	1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	5	900.	.110	.05											
	0.	1.	.09	.10	.79	120.	1.	.09	.10	.79	240.	1.	.09	.10	.79
	360.	3.	.03	.03	.79	480.	5.	.018	.020	.92	600.	7.	.013	.015	1.11
	720.	5.	.018	.020	.42	840.	3.0	.03	.03	1.9	960.	1.	.09	.10	1.85

Fig. C-3. An example of data Set 3 for sewer water quality model

Card	Column	Format	Entry
1-1	21-25	I5	Identification number of the node representing the system outlet; e.g., 8
1-1	26-30	F5.0	Time interval in sec at which values of the direct inflow rate of the hydrographs at the sewer nodes are provided as the input data; e.g., 120.0 sec
1-1	31-80	Blank	

Three examples of data card Set 1 are given in Fig. C-1.

Set 2. Sewer Data — This set consists of as many subsets as the total number of sewers in the system (minimum 1, maximum 100). Each subset corresponds to a sewer. The order of the subsets within Set 2 is arbitrary. Each subset contains 2 to 35 data cards depending on the number of data in the sewer hydrograph. The ordinates of the sewer hydrographs may be given at irregular time intervals. However, the time values should be in increasing order.

Card	Column	Format	Entry
2-1	1-5	I5	Identification number of the junction node at the upstream end of the sewer; e.g., 10
2-1	6-10	I5	Identification number of the junction node at the downstream end of the sewer; e.g., 15
2-1	11-15	F5.0	Length of the sewer in ft or m; e.g., 400.0 ft
2-1	16-20	F5.0	Diameter of the sewer in ft or m; e.g., 1.75 ft
2-1	21-25	F5.0	Initial concentration of the first pollutant at the upstream end of the sewer in ppm or mg/l; e.g., 0.001 ppm
2-1	26-30	F5.0	Initial concentration of the first pollutant at the downstream end of the sewer in ppm or mg/l; e.g., 0.05 ppm
2-1	31-35	F5.0	Initial concentration of the second pollutant at the upstream end of the sewer; e.g., 0.002 ppm
2-1	36-40	F5.0	Initial concentration of the second pollutant at the downstream end of the sewer; e.g., 0.06 ppm
2-1	41-80	Blank	

Card	Column	Format	Entry
2-2	1-6	F6.0	Time of the first discharge and depth data of the hydrograph in sec; e.g., 0.0 sec
2-2	7-11	F5.0	Discharge at sewer entrance at the time given in Col. 1-6 in cfs or m^3/sec ; e.g., 1.0 cfs
2-2	12-16	F5.0	Flow depth at sewer entrance at the time given in Col. 1-6 in ft or m; e.g., 0.4 ft
2-2	17-21	F5.0	Discharge at sewer exit at the time given in Col. 1-6 in cfs or m^3/sec ; e.g., 1.0 cfs
2-2	22-26	F5.0	Flow depth at sewer exit at the time given in Col. 1-6 in ft or m; e.g., 0.88 ft
2-2	27-32	F6.0	Time of the second discharge and depth data of the hydrograph in sec; e.g., 210.0 sec
2-2	33-37	F5.0	Discharge at sewer entrance at the time given in Col. 27-32 in cfs or m^3/sec ; e.g., 4.5 cfs
2-2	38-42	F5.0	Flow depth at sewer entrance at the time given in Col. 27-32 in ft or m, e.g., 0.8 cfs
2-2	43-47	F5.0	Discharge at sewer exit at the time given in Col. 27-32 in cfs or m^3/sec , e.g., 2.8 cfs
2-2	48-52	F5.0	Flow depth at sewer exit at the time given in Col. 27-32 in ft or m; e.g., 0.97 ft
2-2	53-58	F6.0	Time for the discharge and depth data of the hydrographs in sec
2-2	59-63	F5.0	Discharge at sewer entrance at the time given in Col. 53-58
2-2	64-68	F5.0	Flow depth at sewer entrance at the time given in Col. 53-58
2-2	69-73	F5.0	Discharge at sewer exit at the time given in Col. 53-58
2-2	74-78	F5.0	Flow depth at sewer exit at the time given in Col. 53-58
2-2	79-80	Blank	
2-3			Repeat the above data card (2-2) for the discharge and stage values for the successive times until the entire discharge and stage hydrographs at the entrance and exit of the sewer are completely described.
(2-35 max)			

Two examples of data Set 2 are given in Fig. C-2.

Set 3. Reservoir-type junction data — This set consists of as many subsets as the total number of reservoir-type junctions in the system (minimum 0, maximum 100). Each subset corresponds to a junction. The order of the subsets within Set 3 is arbitrary. Each subset contains 2 to 35 data cards depending on the number of data in the input pollutographs and hydrographs.

Card	Column	Format	Entry
3-1	1-5	I5	Identification number of the junction (node); e.g., 5
3-1	6-10	F5.0	Cross-sectional area of the junction in sq ft or sq m; e.g., 900 sq ft
3-1	11-15	F5.0	Initial concentration of the first pollutant in the junction in ppm or mg/l; e.g., 0.10 ppm
3-1	16-20	F5.0	Initial concentration of the second pollutant in the junction in ppm or mg/l; e.g., 0.05 ppm
3-1	21-80	Blank	
3-2	1-6	F6.0	Time of the first data of the direct inflow in sec; e.g., 0.0 sec
3-2	7-10	F4.0	Discharge of the direct inflow at the time given in Col. 1-6 in cfs or m ³ /sec; e.g., 1.0 cfs
3-2	11-14	F4.0	Concentration of the first pollutant of the direct inflow at the time given in Col. 1-6 in ppm or mg/l; e.g., 0.09 ppm
3-2	15-18	F4.0	Concentration of the second pollutant of the direct inflow at the time given in Col. 1-6 in ppm or mg/l; e.g., 0.10 ppm
3-2	19-22	F4.0	Water stage in the junction at the time given in Col. 1-6 in ft or m; e.g., 0.79 ft
3-2	23-28	F6.0	Time of the second data of the direct inflow in sec; e.g., 120.0 sec
3-2	29-32	F4.0	Discharge of the direct inflow at the time given in Col. 23-28 in cfs or m ³ /sec; e.g., 1.0 cfs

Card	Column	Format	Entry
3-2	33-36	F4.0	Concentration of the first pollutant of the direct inflow at the time given in Col. 23-28 in ppm or mg/l; e.g., 0.09 ppm
3-2	37-40	F4.0	Concentration of the second pollutant of the direct inflow at the time given in Col. 23-28 in ppm or mg/l; e.g., 0.10 ppm
3-2	41-44	F4.0	Water stage in the junction at the time given in Col. 23-28 in ft or m; e.g., 0.79 ft
3-2	45-50	F6.0	Time of data in sec; e.g., 240.0 sec
3-2	51-54	F4.0	Discharge of the direct inflow at the time given in Col. 45-50 in cfs or m ³ /sec; e.g., 1.0 cfs
3-2	55-58	F4.0	Concentration of the first pollutant of the direct inflow at the time given in Col. 45-50 in ppm or mg/l; e.g., 0.09 ppm
3-2	59-62	F4.0	Concentration of the second pollutant of the direct inflow at the time given in Col. 45-50 in ppm or mg/l; e.g., 0.10 ppm
3-2	63-66	F4.0	Water stage in the junction at the time given in Col. 45-50 in ft or m; e.g., 0.79 ft
3-2	67-80	Blank	
3-3			Repeat the above data card (3-2) for each reservoir-type junction until the junction direct inflow hydrograph is completely accounted for.
.			
.			
(3-35 max)			

The hydrograph ordinates should be given in equal time intervals on Card 3-2 as obtained from the output of the Illinois Surface Runoff Model computer program. An example of data Set 3 is given in Fig. C-3.

A schematic drawing of the sequence of the input card decks is shown in Fig. C-4.

(b) Printed Output

The printed output of the Illinois Sewer System Water Quality Model gives the values of the pollutant concentrations in each of the reservoir-type junctions, and also at the sewer system outlet, at equal time intervals

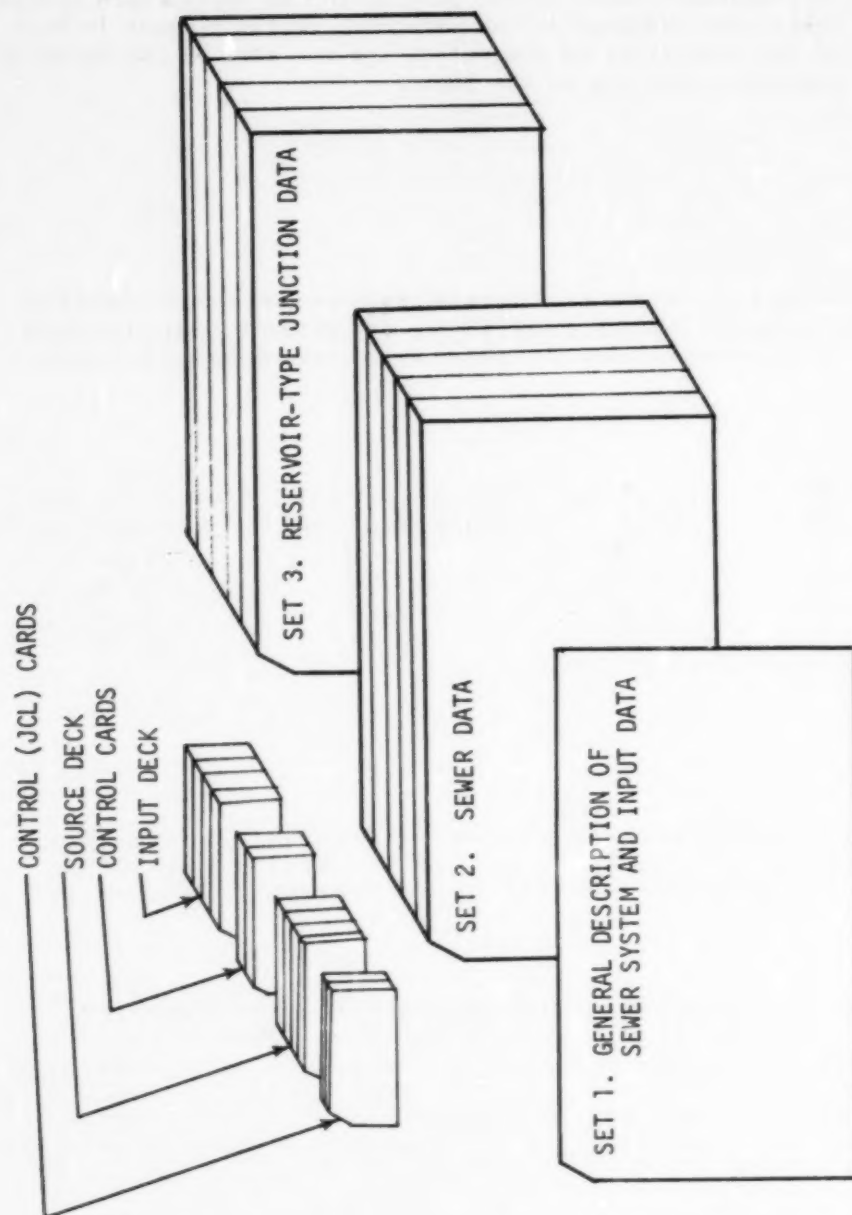


Fig. C-4. Input card deck arrangement for sewer water quality model

specified in Col. 26-30 on Card 1-1. A sample printout is shown in Fig. C-5 for English units. When SI units are used, the user can either change the printout statement in the program to the appropriate headings or simply ignore the headings in the printout, realizing that in fact the units of the quantities in the output are the same as the units of the corresponding quantities in the input.

 WATER QUALITY CONDITIONS AT SEWER SYSTEM OUTLET*****

TIME (SEC)	DISCHARGE (CFS)	1ST POLLUTANT CONCENTRATION	2ND POLLUTANT CONCENTRATION
120.00	1.80	0.08	0.40
240.00	3.20	0.08	0.38
360.00	2.40	0.07	0.36
480.00	1.20	0.07	0.34

 CONDITIONS AT JUNCTION NO= 110*****

TIME (SEC)	STORAGE (FT3)	1ST POLLUTANT CONCENTRATION	2ND POLLUTANT CONCENTRATION
120.00	480.00	0.01	0.05
180.00	540.00	0.02	0.12
360.00	480.00	0.04	0.22
480.00	400.00	0.05	0.34

Fig. C-5. Sample printout of sewer water quality model

(c) Program Listing

```

C
C      SEWER SYSTEM WATER QUALITY MODEL
C
1      DIMENSION QQ1(100),QQ2(100),Q11(100),Q12(100)
2      DIMENSION QOUT1(100,100),QOUT2(100,100),QIN1(100,100),QIN2(100,100
*)
3      DIMENSION TYM(100),QU(100),HU(100),QD(100),HD(100)
4      DIMENSION QQU(100),QQD(100),HHU(100),HHD(100),AAU(100),AAD(100)
5      DIMENSION QN(100),HN(100),NODE(100)
6      DIMENSION AREA(100),C1IN(100),C2IN(100)
7      DIMENSION NODEUP(100),NODOWN(100),PLENGT(100),D(100),C1U(100)
8      DIMENSION C1D(100),CN1(100),CN2(100)
9      DIMENSION C2U(100),C2D(100),Y(100),TM(100)
C
C      FOLLOWING ARE THE GENERAL INPUT PARAMETERS
C
C      NPIPE,TOTAL NUMBER OF SEWERS IN THE SYSTEM
C      NSTJUN,TOTAL NUMBER OF RESERVOIR-TYPE JUNCTIONS IN THE SYSTEM
C      NPYPDB,NUMBER OF POINTS USED TO DESCRIBE EACH OF THE DISCHARGE AND
C      STAGE HYDROGRAPHS AT THE ENTRANCE AND EXIT OF EACH SEWER
C      NJNCDB,NUMBER OF POINTS USED TO DESCRIBE DIRECT INFLOW HYDROGRAPHS
C      AT EACH OF THE SEWER NODES
C      NROOT,IDENTIFICATION NUMBER OF THE SEWER NODE AT THE SYSTEM OUTLET
C      DT,THE TIME INTERVAL AT WHICH ORDINATES OF THE DIRECT INFLOW
C      HYDROGRAPHS AT THE SEWER NODES ARE PROVIDED
C
10     READ(5,1)NPIPE,NSTJUN,NPYPDB,NJNCDB,NROOT,DT
11     FORMAT(5I5,F5.0)
C
C      FOLLOWING ARE THE INPUT PARAMETERS FOR EACH SEWER
C
C      NODEUP(I),IDENTIFICATION NO OF JUNCTION NODE UPSTREAM OF SEWER I
C      NODOWN(I),IDENTIFICATION NO OF JUNCTION NODE DOWNSTREAM OF SEWER I
C      PLENGT(I),LENGTH OF SEWER I
C      D(I),DIAMETER OF SEWER I
C      C1U(I),INITIAL CONCENTRATION OF THE FIRST POLLUTANT AT ENTRANCE OF I
C      C1D(I),INITIAL CONCENTRATION OF THE FIRST POLLUTANT AT EXIT OF I
C      C2U(I),INITIAL CONCENTRATION OF THE SECOND POLLUTANT AT ENTRANCE OF I
C      C2D(I),INITIAL CONCENTRATION OF THE SECOND POLLUTANT AT EXIT OF I
C      TYM(J1),TIME AT J1'ST TIME LEVEL
C      QU(J1),DISCHARGE AT ENTRANCE AT J1'ST TIME LEVEL
C      HU(J1),FLOW DEPTH AT ENTRANCE AT J1'ST TIME LEVEL
C      QD(J1),DISCHARGE AT EXIT AT J1'ST TIME LEVEL
C      HD(J1),FLOW DEPTH AT EXIT AT J1'ST TIME LEVEL
C
12     DO 2 I=1,NPIPE
13     READ(5,3)NODEUP(I),NODOWN(I),PLENGT(I),D(I),C1U(I),C1D(I),C2U(I),C
*2D(I)
14     IF(C1D(I).EQ.C1U(I))C1D(I)=1.05*C1U(I)
15     IF(C2D(I).EQ.C2U(I))C2D(I)=1.05*C2U(I)
16     3 FORMAT(2I5,6F5.0)
17     READ(5,4)(TYM(J1),QU(J1),HU(J1),QD(J1),HD(J1),J1=1,NPYPDB)
18     4 FORMAT(F6.0,4F5.0,F6.0,4F5.0,F6.0,4F5.0)
19     IF(NODOWN(I).NE.NROOT)GO TO 230
20     WRITE(6,74)
21     WRITE(6,73)
22     73 FORMAT(1X,'WATER QUALITY CONDITIONS AT SEWER SYSTEM OUTLET*****')
23     WRITE(6,74)
24     74 FORMAT(1X,'*****')

```

```

WRITE(6,75)
75 FORMAT(///4X,'TIME',5X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POL
*LUTANT')
WRITE(6,75)
76 FORMAT(3X,'(SEC)',6X,'(CFS)',6X,'CONCENTRATION',3X,'CONCENTRATION'
*)
GO TO 900
66 FORMAT(F7.2,5X,F8.2,9X,F6.2,9X,F6.2)
230 WRITE(6,74)
WRITE(6,233) NODEUP(I), NCDOWN(I)
233 FORMAT(1X,'CONDITIONS IN SEWER FROM NODE',I5,'TO NODE',I5,'*****')
WRITE(6,74)
WRITE(6,234)
234 FORMAT(//6X,'AT UPSTREAM SECTION',45X,'AT DOWNSTREAM SECTION')
WRITE(6,235)
235 FORMAT(2X,'TIME',5X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POLLUTA
*NT',18X,'DISCHARGE',3X,'1ST POLLUTANT',3X,'2ND POLLUTANT')
WRITE(6,236)
236 FORMAT(1X,'(SEC)',6X,'(CFS)',6X,'CONCENTRATION',3X,'CONCENTRATION'
*,20X,'(CFS)',5X,'CONCENTRATION',3X,'CONCENTRATION')
900 DO 6 LIN=1,4
GO TO(7,8,9,10),LIN
7 DO 11 K1=1,NPYPDB
11 Y(K1)=QU(K1)
GO TO 15
8 DO 12 K1=1,NPYPDB
12 Y(K1)=HU(K1)
GO TO 15
9 DO 13 K1=1,NPYPDB
13 Y(K1)=QD(K1)
GO TO 15
10 DO 14 K1=1,NPYPDB
14 Y(K1)=HD(K1)
15 K=1
TE=DT
17 K=K+1
J2=2
69 IF(TE-TYM(J2))112,111,110
110 J2=J2+1
GO TO 69
111 YE=Y(J2)
GO TO 200
112 YE=Y(J2-1)+(Y(J2)-Y(J2-1))/(TYM(J2)-TYM(J2-1))*(TE-TYM(J2-1))
200 TM(K)=TE
GO TO(21,22,23,24),LIN
21 QQU(K)=YE
22 HHU(K)=YE
23 QQD(K)=YF
24 HHD(K)=YE
TE=TE+DT
IF(TE.GT.TYM(NPYPDB))KKK=K
IF(TE.GT.TYM(NPYPDB))GO TO 6
GO TO 17
6 CONTINUE
TM(1)=TYM(1)
QQU(1)=QU(1)
QQD(1)=QD(1)
HHU(1)=HU(1)
HHD(1)=HD(1)
DO 80 KK=1,KKK

```

```

THU=2.0*ARCOS(1.-2.*HHU(KK)/D(I))
AAU(KK)=(THU-SIN(THU))*D(I)*D(I)/8.
THD=2.0*ARCOS(1.-2.*HHD(KK)/D(I))
AAD(KK)=(THD-SIN(THD))*D(I)*D(I)/8.
80 CONTINUE
DO 30 L=2,KKK
QA=QQU(L-1)
AA=AAU(L-1)
QB=QQD(L-1)
AB=AAC(L-1)
QDD=QQU(L)
AD=AAU(L)
QC=QQD(L)
AC=AAD(L)
DO 31 LL=1,2
GO TO(41,42),LL
41 IF(L.NE.2)GO TO 43
CA=C1U(I)
CB=C1D(I)
GO TO 45
43 CA=C1A
CB=C1B
GO TO 45
42 IF(L.NE.2)GO TO 44
CA=C2U(I)
CB=C2D(I)
GO TO 45
44 CA=C2A
CB=C2B
45 X1=AC/DT+QC/PLENGT(I)
X2=QDD/PLENGT(I)
B1=-QC/PLENGT(I)
B2=AD/DT-QDD/PLENGT(I)
C=AC*CB/DT
E=AD*CA/DT
CC=(C*B2-E*B1)/(X1*B2-X2*B1)
CD=(X1*E-C*X2)/(X1*B2-X2*B1)
IF(CC.LT.0.)CC=0.
IF(CD.LT.0.)CD=0.
GO TO(51,52),LL
51 QOUT1(I,L)=CD*QDD
DIS1=CC*QC
TIS1=CD*QDD
C1A=CD
C1B=CC
QIN1(I,L)=CC*QC
GO TO 31
52 QOUT2(I,L)=CD*QDD
DIS2=CC*QC
TIS2=CD*QDD
C2A=CD
C2B=CC
63 QIN2(I,L)=CC*QC
31 CONTINUE
P1=DIS1/QC
P2=DIS2/QC
T1=TIS1/QDD
T2=TIS2/QDD
IF(NODOWN(I).NE.NROOT)GO TO 888
WRITE(6,66)TM(L),QC,P1,P2
GO TO 30
888 CONTINUE
237 WRITE(6,533)TM(L),QC,P1,P2,QDD,T1,T2
533 FORMAT(2F10.2,2X,F10.2,9X,F10.2,15X,F10.2,2X,F10.2,9X,F10.2)

```



```

30 CONTINUE
2 CONTINUE
IF(NSTJUN.EQ.0)GO TO 7777
DO 50 JJ=1,NSTJUN

C
C   FOLLOWING ARE THE INPUT PARAMETERS FOR EACH RESERVOIR-TYPE JUNCTION
C
C   NODE(JJ),IDENTIFICATION NUMBER OF THE JUNCTION JJ
C   AREA(JJ),CROSS SECTIONAL AREA OF THE JUNCTION JJ
C   C1IN(JJ),INITIAL CONCENTRATION OF THE FIRST POLLUTANT IN JUNCTION JJ
C   C2IN(JJ),INITIAL CONCENTRATION OF THE SECOND POLLUTANT
C   TYM(JJ),TIME AT J3'RD TIME LEVEL
C   QN(JJ),RATE OF DIRECT INFLOW AT J3'RD TIME LEVEL
C   CN1(JJ),CONCENTRATION OF THE FIRST POLLUTANT OF DIRECT INFLOW
C   CN2(JJ),CONCENTRATION OF THE SECOND POLLUTANT OF DIRECT INFLOW
C   HNI(JJ),WATER STAGE IN JUNCTION AT J3'RD TIME LEVEL
C
  READ(5,151)NODE(JJ),AREA(JJ),C1IN(JJ),C2IN(JJ)
151 FORMAT(15,3F5.0)
  READ(5,152)(TYM(JJ),QN(JJ),CN1(JJ),CN2(JJ),HNI(JJ),J3=1,NJNCD8)
152 FORMAT(F6.0,4F5.0,F6.0,4F5.0,F6.0,4F5.0)
  WRITE(6,74)
  WRITE(6,54)NODE(JJ)
54 FORMAT(1X,'CONDITIONS AT JUNCTION NO=',15,'*****')
  WRITE(6,74)
  WRITE(6,55)
55 FORMAT(///2X,'TIME',6X,'STORAGE',4X,'1ST POLLUTANT',3X,'2ND POLLUT
*ANT')
  WRITE(6,56)
56 FORMAT(1X,'(SEC)',6X,'(FT3)',6X,'CONCENTRATION',3X,'CONCENTRATION'
*)
  IF(KKK.GT.NJNCD8)KKK=NJNCD8
  DO 90 I=1,KKK
    QO1(I)=0.
    QO2(I)=0.
    QI1(I)=0.
90 QI2(I)=0.
    DO 91 MM=1,NPIPE
      IF(NODE(JJ).NE.NODEUP(MM))GO TO 92
      DO 93 L1=2,KKK
        QO1(L1)=QO1(L1)+QOUT1(MM,L1)
        QO2(L1)=QO2(L1)+QOUT2(MM,L1)
93 QO2(L1)=QO2(L1)+QOUT2(MM,L1)
      IF(NODE(JJ).NE.NODDOWN(MM))GO TO 91
      DO 94 L2=2,KKK
        QI1(L2)=QI1(L2)+QIN1(MM,L2)
        QI2(L2)=QI2(L2)+QIN2(MM,L2)
94 QI2(L2)=QI2(L2)+QIN2(MM,L2)
91 CONTINUE
      DO 57 K=2,KKK
        IF(K.EQ.2)C1OLD=C1IN(JJ)
        IF(K.EQ.2)C2OLD=C2IN(JJ)
        BET1=QI1(K)+QN(K)*CN1(K)-QO1(K)
        BET2=QI2(K)+QN(K)*CN2(K)-QO2(K)
        HNEW=HNI(K)
        HOLD=HNI(K-1)
        CNEW1=(C1OLD*HOLD+BET1*DT/AREA(JJ))/HNEW
        CNEW2=(C2OLD*HOLD+BET2*DT/AREA(JJ))/HNEW
        IF(CNEW1.LT.0.)CNEW1=0.
        IF(CNEW2.LT.0.)CNEW2=0.
        S=HNEW*AREA(JJ)
        WRITE(6,66)TYM(K),S,CNEW1,CNEW2
        C1OLD=CNEW1
        C2OLD=CNEW2
57 CONTINUE
50 CONTINUE
7777 STOP
END

```

APPENDIX D

APPLICATION OF ILLINOIS URBAN STORM RUNOFF METHOD TO A HYPOTHETICAL BASIN

The Illinois Urban Storm Runoff (IUSR) method has been described in Sec. IX-8 and Sections VI, VII and VIII and the computer programs in App. B and C and Sevuk et al. (1973). The Illinois Surface Runoff Model and the ISS Model of the IUSR method have been applied to the Chicago Oakdale Avenue Drainage Basin for runoff quantity evaluation and compared to other urban storm runoff prediction methods as discussed in Section X. In this appendix the procedure of using the IUSR method for storm runoff quantity prediction is outlined and application of the method to a hypothetical basin is given as an example.

(a) Procedure of Model Implementation

The following is a brief outline of the recommended procedure for implementation of the IUSR method for storm runoff quantity prediction.

1. Reading and understanding of the method — The potential user is urged to familiarize himself with Sections VI and VII and App. B in this report and the ISS Model user's manual (Sevuk et al., 1973).
2. Preparation of hyetographs — Determine the temporal and areal (if any) variations of the rainfall which will be used as the input into the surface runoff model. These hyetographs are either given input from the data sources or obtained through rainfall frequency-intensity-duration-basin size analysis. A possible approach has been described in Sec. IV-1 and App. A. The hyetograph data are prepared as Set 3 of the input data cards of the Illinois Surface Runoff Model as described in App. B(a). A maximum of five hyetographs, one for each rain-zone within the basin is allowed to account for the areal distribution of rainfall.
3. Drainage basin surface characteristics — Prepare the values of parameters describing the characteristics of the surface of the drainage basin. The information needed include

Inlets: type, width and length or equivalent dimension, ratio of opening area to total area, water carry-over factor, location as related to gutters and sewer nodes, and total number of inlets;

Gutters: type, cross sectional shape and dimensions, length, slope, Manning's n , location as related to rain-zone, identification of connecting inlet, and total number of gutters;

Subcatchment strips: length, slope, surface roughness k , width, and location as related to gutters;

Street pavement: length, slope, surface roughness k , width, and location as related to gutters.

Preparation of the input cards for these data is described as Sets 2 and 4 in App. B(a).

4. Infiltration parameters — Prepare values of Horton's infiltration parameters, f_0 , f_c , and k (Eq. 34) for each of the subcatchment strips, street pavement, and gutters. Preparation of the input cards for these data is described as Set 4 in App. B(a).

5. Sewer System Characteristics — The information needed on sewer system includes

Sewer network: data describing the node-link relationship of the sewer system;

Sewer properties: length, slope, shape, size, and surface roughness k of each of the sewers;

Junctions: type of sewer junctions (point or reservoir), their individual node numbers and connecting inlets, cross-sectional area of reservoir-type junctions;

System outlet: hydraulic properties of the system outlet and its node number.

Preparation of the sewer node data for the Illinois Surface Runoff Model is described as Set 5 in App. B(a). Preparation of the sewer system data for the ISS Model is described in the user's manual (Sevuk et al., 1973).

6. Determination of assigned values for the Illinois Surface Runoff Model — The potential user should determine, in view of the particular application, the following assigned values.

- . number of computational grid points for gutter flow routing (Col. 6-10 of Card 4-1, App. B(a));
- . number of computational grid points for subcatchment strip overland flow routing (Col. 46-50, Card 4-4, App. B(a));
- . time interval of computation in min (Col. 21-30, Card 2-1, App. B(a));
- . time in min when the computation should be terminated (Col. 31-40, Card 2-1, App. B(a));
- . time interval of printout (Col. 16-20, Card 2-1, App. B(a));
- . kinematic viscosity of water for the temperature measured or assumed;
- . initial depth or base flow in sewers, gutters, subcatchment strips, and street pavement. Even if the flow actually starts with a dry bed, a negligibly small but non-zero initial depth or base flow should be assumed to avoid computational singularity;
- . identification of imaginary inlets and gutters.

7. Preparation of input data card decks — Prepare the input data card decks following the instruction for the Illinois Surface Runoff Model and for the ISS Model separately. The input data card decks for the former should follow the order shown in Fig. B-5 and that for the ISS Model in Fig. 17 of Sevuk et al. (1973).

8. Execution of the Illinois Surface Runoff Model — The output cards of the time-discharge values are used as part of the input to the ISS Model.
9. Execution of the ISS Model — With the data cards and output cards from the Illinois Surface Runoff Model, the hydrographs for the sewers of the system are obtained through the execution of the ISS Model. The procedure is described in Sevuk et al. (1973).

(b) Hypothetical Drainage Basin

The hypothetical drainage basin is a 11-hectare (27-ac) area which has been used elsewhere (Yen et al., 1974) to demonstrate a new method for risk-based optimal cost design of storm sewer systems. The drainage pattern of the example basin is shown in Fig. D-1. It consists of 7 subcatchments, 8 gutters, 9 inlet catch basins, 4 sewer junctions and manholes, 4 sewer pipes, and the outlet.

The geometric and hydraulic characteristics of the subcatchments are summarized in Table D-1. Except Subcatchment IV each of the subcatchments has homogeneous drainage characteristics delivering water into a single gutter. In order to demonstrate the capability of the Illinois Surface Runoff Model in handling nonhomogeneous urban blocks, Subcatchment IV is assumed to consist of two parts each draining to a different gutter. The geometric properties of the gutters are summarized in Table D-2. For all the gutters, alleys, and street pavement, the Horton's infiltration parameters (Eq. 34) are assumed to be $f_c = 0.010$ in./hr or 0.254 mm/hr, $f_i = 0.001$ in./hr or 0.025 mm/hr, and $k = 15 \text{ hr}^{-1}$. All the inlets considered are rectangular grate inlets with parallel bars (Type = 1.0) having the length equal to the width = 2.0 ft or 0.61 m and an opening ratio = 0.60.

For the sewer system the characteristics of the four circular sewer pipes are summarized in Table D-3. The four manholes are assumed to have relatively small storage capacities and hence acceptable as point-type sewer junctions. The system outlet has no flow control facility.

(c) Hyetograph for the Hypothetical Basin

In order to demonstrate how the rainfall frequency analysis discussed in Sec. IV-1 can be applied to urban drainage problems, it is assumed that the Urbana, Illinois hourly precipitation data discussed in Sec. IV-1 as an example is applicable to the hypothetical drainage basin being considered here. In order to illustrate the capability of the Illinois Surface Runoff Model to handle successive rainstorms, it is assumed in this example that the rainfall input consists of two rainstorms 5 min apart, the first one having an exceedance probability of 0.23, and the second one having an exceedance probability of 0.37. From Fig. 10 the rainstorm depth, D , corresponding to a 0.23 exceedance probability, or 0.77 non-exceedance probability, is 0.4 in. (10 mm). Subsequently, from

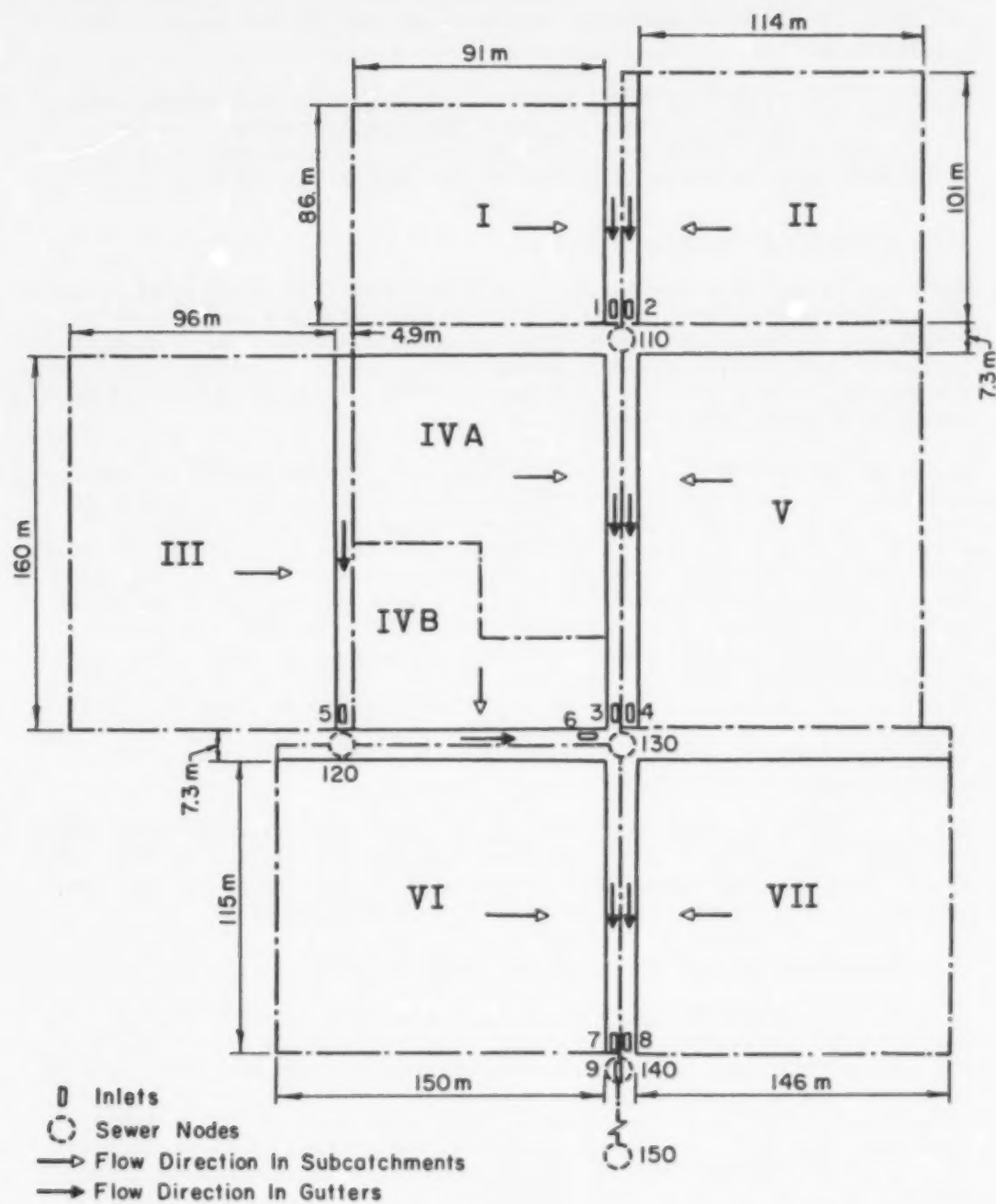


Fig. D-1. Hypothetical drainage basin

Table D-1. CHARACTERISTICS OF SUBCATCHMENTS OF EXAMPLE BASIN

Subcatchment	Area, A		Ground slope, S_o	Surface roughness, k	Infiltration parameters		
	ac	m^2			f_o	f_c	k
					mm/hr	mm/hr	hr ⁻¹
I	2.0	8,090	0.057	0.0024	37.0	12.3	5
II	3.0	12,140	0.057	0.0024	37.0	12.3	5
III	4.0	16,190	0.060	0.0036	37.5	12.6	10
IV-A	2.6	10,520	0.057	0.0024	37.5	12.6	10
IV-B	1.4	5,710	0.062	0.0046	37.5	12.6	10
V	5.0	20,230	0.060	0.0030	38.1	12.7	15
VI	4.5	18,210	0.060	0.0046	38.1	12.7	15
VII	4.5	18,210	0.060	0.0046	38.1	12.7	15

Table D-2. PHYSICAL CHARACTERISTICS OF GUTTERS OF EXAMPLE BASIN

Gutter	Length of gutter		Type of gutter	Longitudinal slope	Contributing street pavement width		Sewer node to which downstream inlet is connected
	ft	m			ft	m	
1	281	86	1.0	0.0081	10	3.0	110
2	330	101	1.0	0.0081	10	3.0	110
3	550	167	1.0	0.0081	10	3.0	130
4	550	167	1.0	0.0081	10	3.0	130
5	526	160	1.0	0.0081	14	4.3	120
6	298	91	1.0	0.0064	10	3.0	130
7	388	119	1.0	0.0064	10	3.0	140
8	400	122	1.0	0.0064	10	3.0	140
(9)	0	0	0.0 (imaginary)	0	0	0	140

Manning's n for all gutters = 0.014

Gutter width for all gutters = 2.0 ft = 0.61 m

Angle with vertical for all gutters = 1.5 rad

Table D-3. CHARACTERISTICS OF SEWERS OF EXAMPLE BASIN

From node	To node	Sewer length		Sewer diameter		Sewer slope	Sewer pipe roughness		Base flow	
		ft	m	ft	m		ft	m	cfs	m ³ /sec
110	130	550	168	2.25	0.69	0.0081	0.005	0.0015	0.5	0.014
120	130	450	137	1.75	0.53	0.0064	0.005	0.0015	0.5	0.014
130	140	400	122	3.00	0.91	0.0064	0.005	0.0015	1.5	0.042
140	150	450	137	3.50	1.07	0.0064	0.005	0.0015	2.0	0.057

Fig. 12 the most likely duration, t_d , for this depth is 2 hr. From Fig. 13 the most frequently occurred value of t for this duration is about 1 hr. Substituting these values of D , t_d , and t into Eqs. 14, 15, and 16 yields the values of triangular hyetograph shape factors a , b , and h , being 1 hr, 1 hr, and 0.4 in., (10 mm), respectively. However, in this illustration, to avoid unnecessary waste of computer time and money, the duration of the first rainstorm was arbitrarily reduced to 12 min while the depth and shape of the hyetograph remain unchanged. The result is the first hyetograph shown in Fig. D-2.

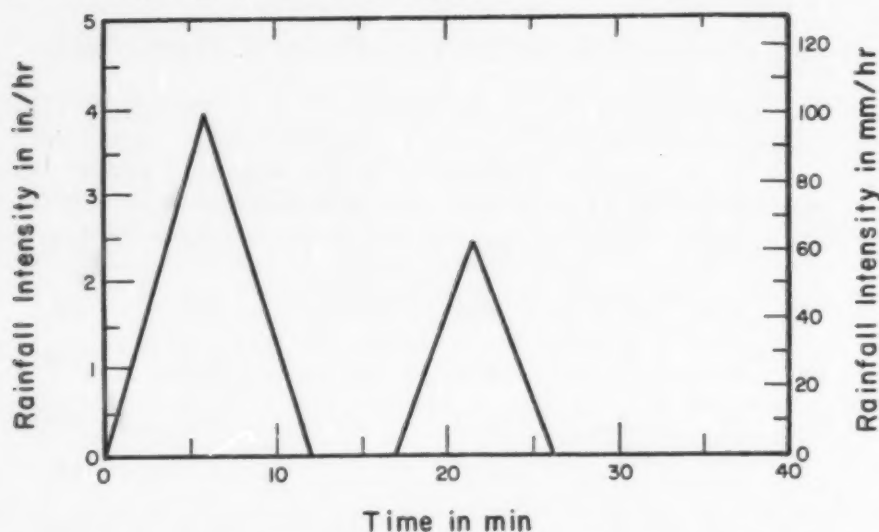


Fig. D-2. Hyetograph for the hypothetical basin

Similarly, for the second rainstorm, the depth corresponding to 0.37 exceedance probability is 0.18 in. (4.6 mm) from Fig. 10. The most likely duration for this depth is 1 hr. Accordingly, t is equal to 0.5 hr, and a and b are both equal to 0.5 hr. From Eq. 16, h is equal to 0.36 in. (9.2 mm). The duration of the second rainstorm was arbitrarily reduced to 9 min to save computer time, while the depth and the shape of the hyetograph were assumed to remain unchanged. The resulting rainfall is shown as the second hyetograph in Fig. D-2.

(d) Computation of Surface Runoff

The surface runoff from the hypothetical basin due to the hyetograph shown in Fig. D-2 is computed by using the Illinois Surface Runoff Model following the procedure outlined in Subsection (b). For the hypothetical basin shown in Fig. D-1, each subcatchment except Subcatchment IV is divided into three rectangular overland flow strips of equal widths and lengths for numerical simulation. Subcatchment IV is divided into seven rectangular strips of various sizes, three of them delivering water into gutter No. 3 whereas the other four strips contributing to the flow in gutter No. 6. Four grid points of computation are used along each subcatchment strip. The initial flow depth in all the subcatchments and gutters is assumed to be 0.0001 m.

It should be noted that although inlet No. 9 is actually not directly connected to any real gutter, an imaginary gutter (Type = 0.0) is introduced for the purpose of computation.

The surface flow computations are performed at two-minute intervals ($\Delta t = 2$ min) and they are terminated at $t = 50$ min. The frequency of printout is selected as 1, i.e., every Δt . The kinematic viscosity of water is taken as $0.93 \text{ mm}^2/\text{sec}$ or $0.00001 \text{ ft}^2/\text{sec}$. The computer execution time is 6 min on an IBM 360/75 system.

A listing of the data cards is given in Fig. D-3.

(e) Computation of Flow in Sewer System

The flow in the sewer system is computed by using the ISS Model. The cards for the sewer node hydrographs obtained from the output of the Illinois Surface Runoff Model together with the data cards of the sewer system are input into the ISS Model. Since node No. 140 is a two-way junction, for the ISS Model overlapping Y-segment scheme an imaginary sewer with an upstream junction node No. 200 is introduced for the computation. For each sewer five computational grid points (i.e., four equal distance intervals) are used along the length of the sewer. The base flow is assumed varying from sewer to sewer as listed in Table D-3. The computer execution time is 1.5 min on an IBM 360/75 system. A listing of the data cards is given in Fig. D-4.

A small part of the printout of the ISS Model for the example is shown in Fig. D-5. The outflow discharge hydrograph of the system is shown in Fig. D-6.

```

2
  9      4      1      1      2.0      50.0      9.81      0.93      24.0
  0.0      27.0      6
0. 0. 6.0 100.0 12. 0. 17. 0. 21.5 60. 20. 0.
  1 4 1 1.0 86. .61 .0081 1.5 1.0 .61 .61 .60 3.0 .0001 .014
.254 .025 15. .254 .025 15. .01 .01

91. .057.0024 37. 12.3 5.0 .0001 .01 .01 4
91. .057.0024 37. 12.3 5.0 .0001 .01 .01 4
91. .057.0024 37. 12.3 5.0 .0001 .01 .01 4
  2 4 1 1.0 101. .61 .0081 1.5 1.0 .61 .61 .60 3.0 .0001 .014
.254 .025 15. .254 .025 15. .01 .01

114. .057.0024 37. 12.3 5.0 .0001 .01 .01 4
114. .057.0024 37. 12.3 5.0 .0001 .01 .01 4
114. .057.0024 37. 12.3 5.0 .0001 .01 .01 4
  3 5 1 1.0 167. .61 .0081 1.5 1.0 .61 .61 .60 3.0 .0001 .014
.254 .025 15. .254 .025 15. .01 .01
      1.0
91. .057.0024 37.5 12.6 10. .0001 .01 .01 4
91. .057.0024 37.5 12.6 10. .0001 .01 .01 4
45.5 .057.0024 37.5 12.6 10. .0001 .01 .01 4
  0.
  4 4 1 1.0 167. .61 .0081 1.5 1.0 .61 .61 .60 3.0 .0001 .014
.254 .025 15. .254 .025 15. .01 .01
  2
114. .060.0030 38.1 12.7 15. .0001 .01 .01 4
114. .060.0030 38.1 12.7 15. .0001 .01 .01 4
114. .060.0030 38.1 12.7 15. .0001 .01 .01 4
  5 4 1 1.0 160. .61 .0081 1.5 1.0 .61 .61 .60 4.3 .0001 .014
.254 .025 15. .254 .025 15. .01 .01

96. .06 .0036 37.5 12.6 10. .0001 .01 .01 4
96. .06 .0036 37.5 12.6 10. .0001 .01 .01 4
96. .06 .0036 37.5 12.6 10. .0001 .01 .01 4
  6 5 1 1.0 91. .61 .0064 1.5 1.0 .61 .61 .60 3.0 .0001 .014
.254 .025 15. .254 .025 15. .01 .01
  5
83.6 .062.0046 37.5 12.6 10. .0001 .01 .01 4
83.6 .062.0046 37.5 12.6 10. .0001 .01 .01 4
41.8 .062.0046 37.5 12.6 10. .0001 .01 .01 4
41.8 .062.0046 37.5 12.6 10. .0001 .01 .01 4
  7 4 1 1.0 119. .61 .0064 1.5 1.0 .61 .61 .60 3.0 .0001 .014
.254 .025 15. .254 .025 15. .01 .01
  3 6
150. .060.0046 38.1 12.7 15. .0001 .01 .01 4
150. .060.0046 38.1 12.7 15. .0001 .01 .01 4
150. .060.0046 38.1 12.7 15. .0001 .01 .01 4
  8 4 1 1.0 122. .61 .0064 1.5 1.0 .61 .61 .60 3.0 .0001 .014
.254 .025 15. .254 .025 15. .01 .01
  4
146. .060.0046 38.1 12.7 15. .0001 .01 .01 4
146. .060.0046 38.1 12.7 15. .0001 .01 .01 4
146. .060.0046 38.1 12.7 15. .0001 .01 .01 4
  9 2 1 0.0 50.
      1.0 1.0

7 8      1.0 1.0

110 2.0 .014 .01 .01 1 2
120 2.0 .014 .01 .01 5
130 1.0 .014 .01 .01 3 4 6
140 1.0 .014 .01 .01 7 8 9

```

Fig. D-3. Input data cards for the example for Illinois Surface Runoff Model

```

START TI=120 GOPARM=* INTVL=10 * NOGRAPH
NODE 110 120 130 140 150 200
IMAGINARY 200
YJUNCT 130 140
ROOT 150
PD 110 130 550 0.0081 2.00 .005 0 110
PD 120 130 450 0.0064 1.75 .005 0 90
PD 130 140 400 0.0064 3.00 .005 0 80
PD 140 150 450 0.0064 3.50 .005 0 90
PD 200 140 0 0 0 0
FBD 110 0 3000 0.5
0 0.5,120 0.52,240 0.82,360 7.46,480 7.06,600 3.01,720 1.65,840 1.04,
960 0.82,1080 0.75,1200 1.51,1320 3.44,1440 2.10,1560 1.24,1680 0.94,
1800 0.78,1920 0.69,2160 0.60,2400 0.57,2640 0.53,2880 0.52,3000 0.50
FEND
JFBD 130 0 3720 0.5
0 0.5,120 0.53,240 0.88,360 11.08,480 12.91,600 7.26,720 4.11,840 2.54,
960 1.78,1080 1.41,1200 2.81,1320 6.26,1440 4.83,1560 3.21,1680 2.17,
1800 1.59,2040 1.01,2280 0.80,2520 0.69,2760 0.63,3000 0.61,3720 0.50
JEND
JFBD 140 0 3720 0.5
0 0.5,120 0.53,240 0.97,360 21.97,480 25.63,600 13.22,720 7.25,840 4.55,
960 3.14,1080 2.40,1200 5.06,1320 7.03,1440 8.56,1560 5.56,1680 3.79,
1800 2.75,2040 1.66,2280 1.17,2520 0.92,2760 0.80,3000 0.72,3720 0.50
JEND
FBD 120 0 3720 0.5
0 0.50,120 0.51,240 0.73,360 5.83,480 6.08,600 2.90,720 1.50,840 1.00,
960 0.78,1080 0.72,1200 1.30,1320 2.84,1440 1.95,1560 1.25,1680 0.91,
1800 0.75,2040 0.62,2280 0.57,3720 0.50
FEND
CEND

```

Fig. D-4. Input data cards for the example for ISS Model

INPUT HYDROGRAPH SPECIFIED FOR NODE # 110

INCREMENT #	ABS. TIME, SEC	DISCHARGE, CFS
0	0.0	0.5
1	120.0	0.5
2	240.0	0.8
3	360.0	7.4
4	480.0	7.0
5	600.0	3.0
6	720.0	1.6
7	840.0	1.0
8	960.0	0.8
9	1,080.0	0.7
10	1,200.0	1.5
11	1,320.0	3.4
12	1,440.0	2.0
13	1,560.0	1.2
14	1,680.0	0.9
15	1,800.0	0.7
16	1,920.0	0.6
17	2,040.0	0.5
18	2,160.0	0.5
19	2,280.0	0.5
20	2,400.0	0.5

INPUT HYDROGRAPH CALCULATED FOR NODE # 130

ABS. TIME, SEC	DEPTH, FEET	DISCHARGE, CFS
120.0	0.38	1.5
240.0	0.45	2.0
360.0	1.27	15.8
480.0	1.71	26.4
600.0	1.34	17.5
720.0	0.96	9.3
840.0	0.74	5.5
960.0	0.61	3.8
1,080.0	0.55	3.0
1,200.0	0.67	4.5
1,320.0	1.00	9.9
1,440.0	1.02	10.4
1,560.0	0.83	6.9
1,680.0	0.67	4.6
1,800.0	0.58	3.4
1,920.0	0.44	2.0

FLOW CONDITIONS AT TIME = 396.2 SECONDS

INTERIOR STATION NUMBER	FROM NODE 110 TO NODE 130	FROM NODE 130 TO NODE 140
0	VELOCITY 4.91 DEPTH 0.96 DISCHARGE 7.3	VELOCITY 6.14 DEPTH 1.49 DISCHARGE 21.5
1	VELOCITY 5.89 DEPTH 0.85 DISCHARGE 7.5	VELOCITY 6.97 DEPTH 1.30 DISCHARGE 20.4
2	VELOCITY 6.02 DEPTH 0.83 DISCHARGE 7.5	VELOCITY 7.06 DEPTH 1.24 DISCHARGE 19.3
3	VELOCITY 5.94 DEPTH 0.81 DISCHARGE 7.1	VELOCITY 6.97 DEPTH 1.17 DISCHARGE 17.8
4	VELOCITY 5.87 DEPTH 0.76 DISCHARGE 6.4	VELOCITY 6.83 DEPTH 1.11 DISCHARGE 16.3
5	VELOCITY 5.64 DEPTH 0.69 DISCHARGE 5.4	VELOCITY 6.65 DEPTH 1.05 DISCHARGE 14.6

Fig. D-5. Sample printout for the example

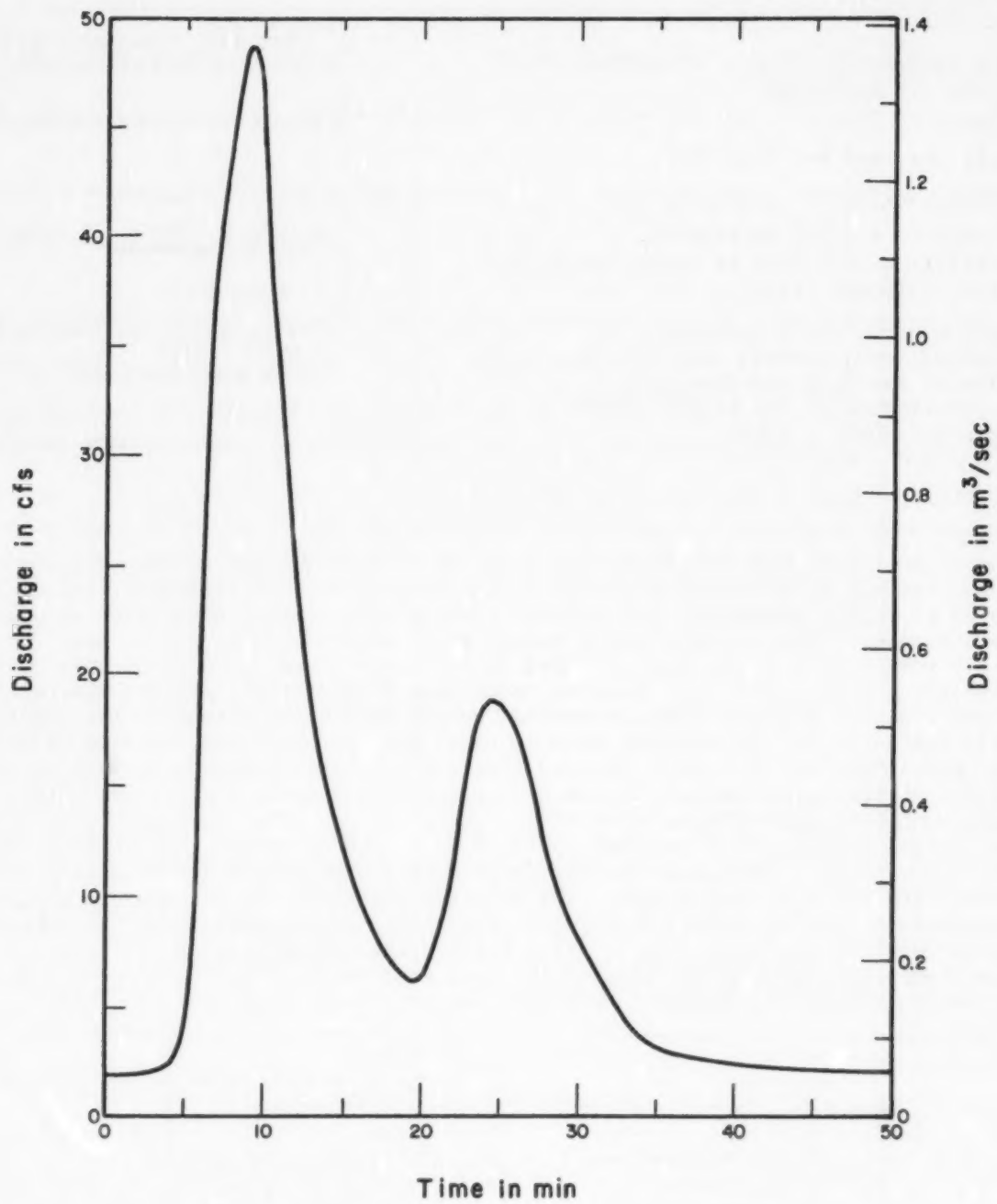


Fig. D-6. Runoff hydrograph for the example